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Diogo Filipe Teixeira Cerqueira da Silva

Parameter analysis of the armour layer in coastal structures

Análise de parâmetros do manto resistente de estruturas costeiras

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O júri / The jury

Presidente / President

Professor Doutor Paulo Barreto Cachim
Professor Associado da Universidade de Aveiro

Vogais / Committee

Doutora Conceição Juana Fortes
Investigadora Principal do Laboratório Nacional de Engnharia Civil

Professor Doutor Carlos Daniel Borges Coelho
Professor Auxiliar da Universidade de Aveiro (orientador)

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Palavras-chave Obra costeira; Dimensionamento; Coeficiente de estabilidade; Formulações empíricas; Análise de sensibilidade.

Resumo

A erosão costeira é um problema grave que afecta muitos países do mundo e em particular, Portugal. O défice sedimentar e a crescente pressão urbanística, aliadas a um regime costeiro energético, anteveem a necessidade de avultados investimentos em estruturas de proteção costeira.

O processo de dimensionamento de estruturas costeiras passa pela utilização de formulações empíricas, seguido de testes em modelo físico para validar as soluções. Nestas formulações, a incorporação de diversos aspectos em coeficientes, adiciona um nível de subjetividade relevante aos resultados.

A intenção deste trabalho é abordar o problema da subjetividade pela análise do coeficiente de estabilidade, presente na fórmula de Hudson. Na fórmula original, este coeficiente exprime a influência de um certo número de parâmetros na estabilidade do manto de proteção de estruturas costeiras. No entanto, existe uma ausência de valores recomendados que tenham em conta alguns parâmetros importantes. Ao amentar o conhecimento sobre os diversos parâmetros que influenciam o coeficiente de estabilidade, é possível alcançar uma maior precisão.

De entre os parâmetros que influenciam a estabilidade, o foco principal da análise é sobre os parâmetros considerados nas fórmulas de Van der Meer (permeabilidade, duração da tempestade, nível de dano e ângulo do talude da estrutura) e no ângulo de incidência da onda sobre a estrutura. Uma análise de sensibilidade foi feita para avaliar a influência de cada parâmetro no valor do coeficiente de estabilidade e na estabilidade final.

Usando dois casos de estudo, foi feita uma comparação do coeficiente de estabilidade obtido na fase de projeto e o coeficiente que resulta dos testes em modelo físico.

Keywords Coastal structure; Design; Stability coefficient; Empirical formulations; Sensitivity analysis.

Abstract Coastal erosion is a serious problem that affects numerous countries and particularly Portugal. The sediment deficit, increasing urbanistic pressure and highly energetic coastal areas anticipate the necessity of large investments in shore protection structures.

The design process of coastal structures is mainly dependent on empirical formulations, followed by tests on physical models to validate the design solutions. In these empirical formulations, the incorporation of several parameters in to coefficients, adds a level of subjectivity that is relevant on the results.

This document intends to address the subjectivity problem through the analysis of the stability coefficient in the Hudson formula. In the original formula, this coefficient expresses the influence of a certain amount of parameters on the armour layer stability of coastal structures. However, there is an absence of recommended values that take into account some important parameters. By increasing the knowledge over the several parameters that influence the stability coefficient, a better accuracy can be achieved.

The main focus is on the parameters considered by the Van der Meer formulations (permeability, storm duration, damage level and slope angle) and on the incidence angle in which the wave attacks the structure.

A sensitivity analysis was performed for various parameters, in order to evaluate the influence of each parameter on the stability coefficient and final stability values.

Using two study cases, a comparison was performed on the design stability coefficient and the coefficient that resulted from physical tests.

Contents

1 Introduction **1**
1.1 Foreword . . . 1
1.2 Motivation . . . 1
1.3 Objectives . . . 2
1.4 Dissertation outline . . . 3

2 Coastal structures **5**
2.1 Types of Coastal Structures . . . 5
2.1.1 Groins . . . 6
2.1.2 Detached breakwaters . . . 7
2.1.3 Rubble-mound breakwaters . . . 8
2.2 Armour layer material . . . 9
2.2.1 Rock . . . 9
2.2.2 Concrete . . . 10
2.3 Stability of Coastal Structures . . . 13

3 Design of coastal structures **17**
3.1 Reference manuals for coastal structures design . . . 17
3.2 Stability evaluation methods . . . 18
3.3 Hudson formula . . . 20
3.4 Van der Meer formulas . . . 22
3.4.1 Deep water conditions . . . 22
3.4.2 Shallow water conditions . . . 24
3.4.3 Concrete armour layer . . . 27
3.5 Summary of the presented stability formulas . . . 28

4 Hudson stability coefficient **31**
4.1 Typical stability coeficient . . . 31
4.2 Interlocking capability . . . 36
4.2.1 Unit shape . . . 36
4.2.2 Unit placement . . . 37
4.2.3 Unit roughness . . . 37
4.3 Types of wave attacking the structure . . . 38

4.4 Seabed slope in front of the structure 39
4.5 Damage criteria 40
4.6 Wave height 42
4.7 Permeability 43
4.8 Wave incidence angle 45
4.8.1 Approaches 45
4.8.2 Discussion 53
4.9 Other 58

5 Sensitivity analyzes **61**
5.1 General considerations 61
5.2 Discussion 64
5.2.1 Permeability 64
5.2.2 Storm duration 66
5.2.3 Damage level 68
5.2.4 Slope angle 70

6 Structural design examples **73**
6.1 Angeiras 73
6.1.1 Local conditions 73
6.1.2 Preliminary design 77
6.1.3 Physical model 81
6.2 Velas harbor 87
6.2.1 Local conditions 88
6.2.2 Preliminary design 90
6.2.3 Physical model 93
6.3 Discussion of the practical examples 95

7 Final remarks **99**
7.1 General comments 99
7.2 Conclusions 99
7.3 Future developments 102

Bibliography **103**

List of Tables

2.1 Concrete armour units 11

3.1 Relation between relative wave height and damage level 21
3.2 Range of validity of parameters in Van der Meer formulas for deep water conditions (CIRIA *et al.*, 2007) 24
3.3 Range of validity of parameters in modified Van der Meer formulas for shallow water conditions 26
3.4 Overview of fields of application of different stability formulas for rock armoured slopes 29

4.1 Comparison between K_D values in USACE (1977) and USACE (1984) for structure trunk 32
4.2 Comparison between K_D values in USACE (1977) and USACE (1984) for structure head 33
4.3 K_D values for various armour units 34
4.4 K_D values for Antifer blocks 34
4.5 Design values of the damage parameter for armourstone in a double layer 41
4.6 Relative damage level for concrete armour units 42
4.7 Characteristic wave height ratios for a sea-state with a Rayleigh distribution of wave heights 43
4.8 Wave obliquity coefficient X for the equivalent wave height ($H_{S,\beta}$) 48
4.9 Wave obliquity coefficient X for armour stability from various authors . . 52
4.10 Wave angle summary table 59

6.1 Angeiras tide levels 75
6.2 Extreme values of wave height 76
6.3 Maximum significant wave heights at different depths 77
6.4 Project wave heights 79
6.5 Geometric and mechanical parameters for design 80
6.6 Stability parameters for each cross section 81
6.7 Mass and weight of individual blocks 83
6.8 Velas Harbor tide levels 89
6.9 Maximun values of wave height 91
6.10 Comparisson between values obtained by diferent destributions for P1 . . 91

6.11 Test program on the Velas harbor physical model 94
6.12 Slope angle modification values in Angeiras 96
6.13 Comparison between calculated and reference weights in Angeiras 97
6.14 Range of K_D values correspondent to the weight results from the design process 97
6.15 Range of K_D values correspondent to the changes made in the structure of the Angeiras breakwater 98

List of Figures

2.1 Beach configuration with groins 6
2.2 Groin with and without downstream seawall 7
2.3 Typical beach configurations with detached breakwaters 7
2.4 Typical beach configurations with nearshore detached breakwaters 8
2.5 Conventional multilayer rubble-mound breakwater 8
2.6 Examples of concrete armour units 12
2.7 Type of structure as a function of N 14

3.1 Optimum combination of first and maintenance costs 22

4.1 Physical model tests to evaluate Antifer blocks placement 35
4.2 Interlocking effect 36
4.3 Interlock effect on complex and bulky armour units 37
4.4 Placement of armourstone 38
4.5 Wave breaking types 39
4.6 Eroded area schematization 41
4.7 Notional permeability factor 45
4.8 Reduction factor for wave obliquity 48
4.9 Reduction factor for wave obliquity 49
4.10 Reduction factor for wave obliquity 51
4.11 Expression for the reduction in required diameter for oblique waves 53
4.12 Rock armour units reduction factor for various authors 54
4.13 Cube units reduction factors by different approaches 55
4.14 Concrete units reduction factors by different authors 55
4.15 Comparison of the reduction factor by CONSULMAR with other authors (rock) 57
4.16 Comparison of the reduction factor by CONSULMAR with other authors (concrete) 57

5.1 K_D *vs* Nominal permeability (P), for different wave heights 65
5.2 K_D *vs* Wave height, for different permeability values 65
5.3 K_D variation for different values of permeability 66
5.4 K_D *vs* Number of waves (N), for different wave heights 67
5.5 K_D *vs* Wave height, for different number of waves 67

5.6 K_D variation for different number of waves 68
5.7 N_s *vs* Damage level (S_d), for different wave heights 69
5.8 N_s *vs* Wave height, for different damage levels 69
5.9 N_S variation for different damage levels 70
5.10 N_s *vs* Slope angle, different wave heights 71
5.11 N_s *vs* Wave height, for different slope angles 71
5.12 N_S variation for different slopes 72

6.1 Map of Portugal with Angeiras beach location 74
6.2 Layout of the Angeiras breakwater 78
6.3 Generic cross-section of the Angeiras breakwater 79
6.4 Test results for Base Solution 85
6.5 Alterations to the structural characteristics of Base Solution 86
6.6 Alterations to structural characteristics of Alternative Solution 1 87
6.7 Map of Azores 88
6.8 Generic cross-section of the structure 92
6.9 Physical model of the extension of the Velas harbor structures 95

Nomenclature

A_e	–	Eroded area
D	–	Characteristic diameter of the structure, armour unit, stone, gravel or sand
D_β	–	Armour unit diameter for oblique waves
$D_\perp$	–	Armour unit diameter for normal waves
D_{n50}	–	Median nominal size
$D_{n50,core}$	–	Median nominal size of core material
$D_\%$	–	Damage in percentage
F	–	Reduction factor
g	–	Gravity acceleration
h	–	Water depth
H	–	Wave height
H_S	–	Significant wave height
$H_{S,\beta}$	–	Equivalent significant wave height
$H_{S,toe}$	–	Significant wave height at the toe of the structure
H_{so}	–	Deep water wave height
$H_{1/3}$	–	Average of highest on third of the waves in a time series
$H_{1/10}$	–	Average of highest 10% of all waves in a time series
$H_{2\%}$	–	Wave height exceeded by 2% of wave heights in a time series
$H_{D=0}$	–	Design wave height corresponding to the no damage condition
K_D	–	Hudson's stability coefficient
L_{om}	–	Length of the offshore wave
n	–	Number of layers
N	–	Number of waves
N_{od}	–	Relative damage level
N_S	–	Stability number
$N_{S,\beta}$	–	Stability number for oblique waves
$N_{S,\perp}$	–	Stability number for normal waves
P	–	Permeability of the core and inner layers
s_{om}	–	Wave steepness

S_d	–	Damage level
S_R	–	Specific gravity of unit
T	–	Wave period
T_m	–	Mean wave period
$T_{m-1.0}$	–	Spectral wave period
W	–	Unit weight
W_β	–	Unit weight for oblique waves
$W_\perp$	–	Unit weight
α	–	Slope angle
Δ	–	Relative mass density
γ_r	–	Specific weight of the material
γ_w	–	Specific weight of water
ρ_r	–	Mass density of the material
ρ_w	–	Mass density of water
θ	–	Structure angle measured from horizontal
ξ_{cr}	–	Critical surf similarity parameter
ξ_m	–	Mean surf similarity parameter
$\xi_{s-1.0}$	–	surf similarity parameter calculated with $T_{m-1.0}$

Acronyms

BSI	–	British Standard Institution
CEM	–	Coastal Engineering Manual
CD	–	Chart Datum
CHL	–	Coastal and Hydraulics Laboratory
CIRIA	–	Construction Industry Research and Information Association
GDP	–	Gross Domestic Product
SPM	–	Shore Protection Manual
USACE	–	United States Corps of Engineers
LNEC	–	Laboratorio Nacional de Engenharia Civil (National Laboratory or Civil Engineering)

Chapter 1

Introduction

1.1 Foreword

Interaction between sea and land is a demanding and interesting field due to the constant change in shoreline induced by natural and anthropogenic factors. This interaction is the study subject of Coastal Engineering.

Coastal Engineering is one of the branches inside Civil Engineering where subjectivity is still a preponderant aspect to consider, *i.e.* the design of coastal structures is more dependent on designer experience than traditional structures. The design process of coastal structures is mainly dependent on empirical formulations (where subjectivity takes a preponderant role), followed by tests on physical models to validate the designed solutions. This could be considered as an iterative process as the physical model test results may lead to changes in the design that need to be tested again. Improvement on the empirical formulations would result in a need for fewer tests, structures that are more cost-effective and less subjectivity on the application of the formulations.

1.2 Motivation

Despite being a small country, Portugal has an extensive shoreline and the majority of the Portuguese population lives relatively close to the coastal area. Therefore, the coastal area is of great importance for Portugal.

One of the most important economic activities in Portugal is tourism. It represents, a large contribution to GDP (Gross Domestic Product) and to global employment (direct and indirect). This activity is highly dependent on maintaining an attractive coastal area for tourists.

As referred in Coelho (2005), coastal erosion is a serious problem in Portugal and can be attributed to a number of reasons, ranging from increased development along the coastline, the series of interventions done on rivers (construction of dams, sand extraction for construction purposes), which, at one time, supplied coastal sediment, and the highly energetic Portuguese coastal area. Together with the increasing urbanistic pressure, this can create great risk for populations and cause material losses. In this context, it becomes necessary to intervene in the stabilization of the shoreline, especially in urbanized and touristic areas. One of the ways to achieve this is through coastal defense structures.

Besides from the residential and recreational functions, coastal areas provide important economic and transport functions through the construction of harbors and marinas (Reeve *et al.*, 2004). These structures require protection from waves and overall energy from the sea, which is achieved through building structures such as breakwaters.

Coastal structures such as breakwaters are relatively costly to build and of great social and economical importance, reinforcing the importance of an adequate preliminary design that allows a more economic and safe performance of the structure, throughout the expected life period.

The economic factor in the construction of breakwaters has an increased influence for countries with a weak economy such as Portugal due to the traditionally high cost of coastal interventions. Improved design procedures would help to lower the costs of interventions and, therefore, help in raising awareness of the government to the necessity of a more preventive approach to the coastal erosion problem. A good indication of this necessity was the large amount of damages caused by the sea on the winter of 2013/2014, to all the Portuguese coast.

1.3 Objectives

In order to address the subjectivity problem, this document aims to study the parameters that are more relevant and how they influence the the stability of coastal structures, specially the Hudson's stability parameter. The particular importance given to this parameter is due to two reasons: The Hudson formula is the more widely used empirical formulation to assess the stability of coastal structures and the Hudson stability coefficient is the main source of subjectivity present in the formula. The influence of each of the studied parameters is assessed by a sensitivity analyzes, using the Van der Meer formulation.

Within the parameters affecting the stability coefficient, the wave attack angle is studied in more detail. Guidance about this parameter is scattered throughout the bibliography and no definitive recommendation is given in the reference manuals.

1.4 Dissertation outline

Chapter 1 presents a general overview of the dissertation, starting with the problematic that motivates the work performed and presented in this document, the personal motivation of the author and the main established goals to attain. Finally, in the current section the document outline is presented, where the author summarizes the contents of each chapter, useful for the reader's understanding of the organization of this document.

Chapter 2 is dedicated to an overview on the different types of existing coastal structures, the different materials that can constitute the armour layer and notions on structural stability.

In chapter 3, a review of the design tools for a coastal structure is presented, including the reference design manuals, evaluation methods and empiric formulations.

Chapter 4 presents an overview of the definition of the Hudson's stability coefficient, including the recommended values given by reference manuals and other authors for different armour units. In this chapter is also included a review of the main parameters affecting the stability coefficient.

In chapter 5, the results and interpretation of a sensitivity analyzes are presented to better understand the influence of the considered parameters in the stability coefficient value and on the overall stability of the armour layer.

Chapter 6 presents two study cases from actual design projects and physical test results provided by CONSULMAR in order to compare the design stability coefficient and the coefficient that resulted from the physical tests.

Chapter 7 indicates the most significant general comments and main conclusions drawn from the work developed. Finally there are pointed the future developments which may be considered to improve the accuracy of the empirical formulations.

Chapter 2

Coastal structures

The planing of coastal structures can be considered for two distinct objectives. Firstly, there are coastal defense structures, designed to stabilize the coast line and protect the coast areas from erosion effects. The second type of coastal structures are the ones design to protect other constructions, such as marinas and harbors, and to provide better shelter conditions for ship navigation in the entrance and inside the docks.

This chapter is intended to provide an overview on the different types of existing coastal structures, considerations on the associated benefits and downsides, the different materials that can constitute the armour layer and notions on structural stability.

2.1 Types of Coastal Structures

It is important to make the distinction between coastal intervention and coastal structures (Lima, 2011). Coastal interventions are all the actions performed by man in order to mitigate the effects of erosion in a particular stretch of coast line. Coastal structures refers to the construction of definitive structures that act on the transport sediment along the coast to mitigate the erosion problem and are also used in the sheltering and stabilization of harbor basins and entrances.

A coastal defense element is any obstacle present in the coast, that protects the coast line against erosion and, accordingly to Alfredini (2005), can be classified as natural or artificial. Natural elements are the ones present in the coast line by default, without human intervention (*e.g.* rocks, beaches, dunes). The artificial elements are all the interventions realized in the coast, which can be temporary (*e.g.* artificial feeding of beaches, sand by-pass, etc.) or definitive (coastal defense structures).

The most important types of coastal structures and their main characteristics and applications are described in the next sections:

2.1.1 Groins

Groins are narrow structures (figure 2.2(a)), usually straight and perpendicular to the shoreline (Burchart and Hughes, 2011c), whose main function is the stabilization of a stretch of coastline. These structures can be used to prevent erosion as well as accretion.

The construction of a groin leads to accretion of beach material on the updrift side and erosion on the downdrift side. Therefore, it is common to build a groin system (figure 2.1) to protect a large stretch of coastline.

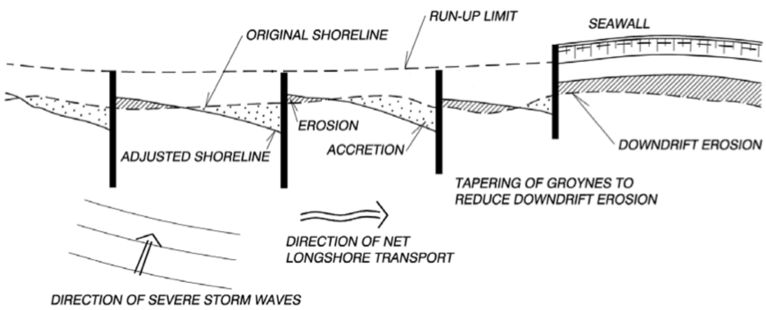


Figure 2.1: Beach configuration with groins (Burchart and Hughes, 2011c)

The effectiveness of a groin solution is highly dependent on the longshore transport. When the longshore transport is week or does not exist, the groin solution should not be adopted (Coelho, 2005).

The orientation, length, height, permeability, and spacing of the groins determine, under given natural conditions, the actual change in the shoreline and the beach level. Because of the potential erosion of the beach, downdrift of the last groin in the field, a transition section of progressively shorter groins may be provided to prevent the formation of a severe erosion area. Even so, it might be necessary to protect some part of the downdrift beach with a seawall (figure 2.2(b)) or to nourish a portion of the eroded area with beach material from an alternative source (Burchart and Hughes, 2011c).

Normally, groins are rubble-mound constructions with an armour layer of stone or concrete units, due to the necessity to withstand severe wave loads.

(a) Simple groin

(b) Groin with seawall

Figure 2.2: Groin with and without downstream seawall (Coelho, 2005)

2.1.2 Detached breakwaters

Detached breakwaters are structures placed parallel to the coast line at relatively high depths, whose function is to reduce the wave force before hitting shore. This reduction in wave force is achieved by means of wave refraction (Marinho, 2013) and leads to accretion on the surf zone (figure 2.3).

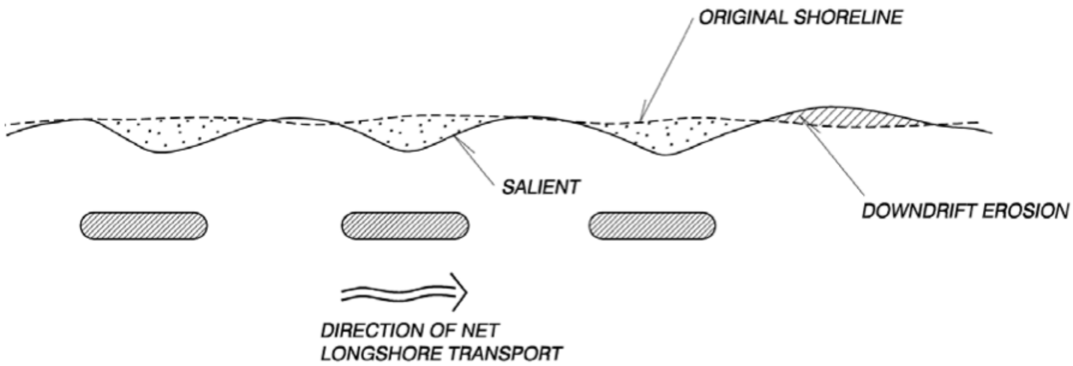


Figure 2.3: Typical beach configurations with detached breakwaters (Burchart and Hughes, 2011c)

If the structures are placed to close to the shoreline, there is the possibility that the sand accretion could form tombolos (figure 2.4). In this case, the detached breakwater starts working similarly to a groin, blocking the longshore transport.

The cross section of the detached breakwaters is similar to rubble-mound groins and are also protected by a layer of armourstone or concrete armour units. Accordingly to (Lima, 2011), the preferred material should be the armourstone due to its roughness and porosity that better dissipates the wave energy. However, since these structures are

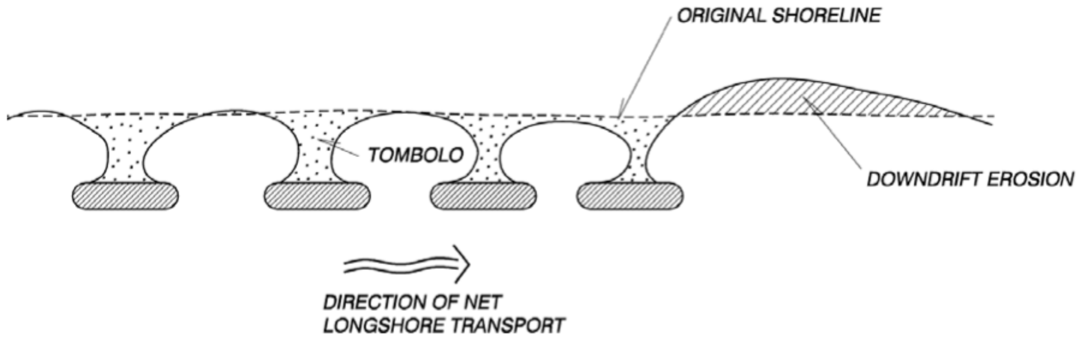


Figure 2.4: Typical beach configurations with nearshore detached breakwaters (Burchart and Hughes, 2011c)

placed at high depths, large quantities of material are required, which sometimes leads to the choice of less expensive concrete blocks.

2.1.3 Rubble-mound breakwaters

Rubble-mound breakwaters are the most common type of breakwater and are constructed with the objective to provide shelter conditions to harbors, marinas and navigation canals from incident waves.

Many shape variations have been idealized. However, the conventional rubble-mound breakwater consist of a core of finer material covered by bigger blocks forming the armour layer. To prevent finer material from being washed out through the armour layer, filter layers must be provided. The filter layer just beneath the armour layer is also called the underlayer (figure 2.5).

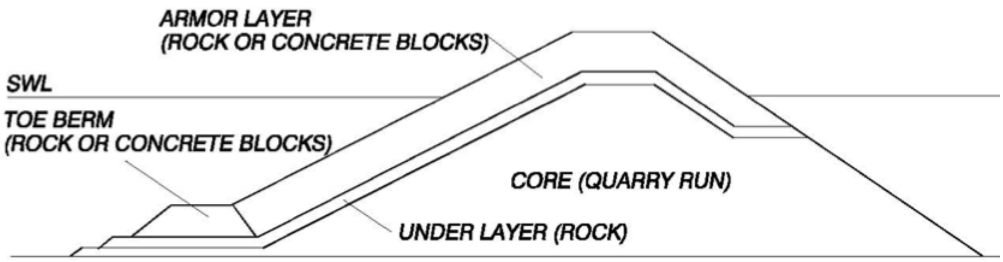


Figure 2.5: Conventional multilayer rubble-mound breakwater (Burchart and Hughes, 2011c)

Material and stability concerns of rubble-mound breakwaters are discussed further in foreword sections.

2.2 Armour layer material

The consideration of the material to use in the armour layer of a breakwater should be addressed at an early stage, before the preliminary design is started. In this stage, account must be taken on the available material sources and transport possibilities, as the material production and transportation costs can be an important consideration when selecting a design solution. In a more advanced stage, a change in the material can be considered to adapt the design to more severe conditions or when the designed solutions proves to be insufficient and the new solutions with the same material are nonexistent or expensive.

2.2.1 Rock

Accordingly to Burchart and Hughes (2011a), rock is used extensively to construct coastal structures, and it is by far the most common material used in the United States for breakwaters, jetties, groins, revetments, and seawalls.

The main considerations for a rock project are the scale, availability, quality and handling of the materials.

The scale of the project is important as while small and medium-size projects can be dependent on existing sources of armourstone, as the project size increases, the importance of an available source of material that is close to the site, increases. Very large projects usually require dedicated quarries to be open within a few kilometers from the site (CIRIA *et al.*, 2007) with all the associated costs and inconvenient.

The availability of rock material is mainly dependent on the maximum stone size that can be produced at a given quarry. Geological expertise are required to locate sources with a desired range of stone size or to predict the best location of large stone sources within a certain distance of the construction site. The geological expertise are also required to evaluate if the quality of the rock meets the design standards.

The handling of the materials can also be a conditioning factor as the means to transport and place armourstones of a certain range of sizes may be unavailable or not cost-effective.

For structures in which the armour layer stones are not bound together, stability is achieved through the relatively high specific weight of stone, assisted to some degree of interlocking, provided by the friction and mechanical interlocking that occurs between adjacent stones. The degree of interlocking is dependent on the stone shape and placement. As for the level of friction between the units, it is dependent on the roughness of the stones.

The selection of armourstones is made by weight to resist a specific wave load. Ideally, all stones are blocky and nearly uniform in size. Burchart and Hughes (2011a) states that the largest stone dimension on an individual stone should be no more than three times the shortest dimension and once the design is complete and stone specific weight has been specified, it is important to ensure stones used in the project meet or exceed the assumed specific weight used in design.

Accordingly to Lima (2011), the main advantages to the use of rock in the armour layer of a breakwater are:

- Durability - rock material is extremely resistant to wave loads;
- Wave energy absorption - the porosity of the rock material, together with the rounded shape, promotes less reflection of the wave forces;
- Flexibility - The rocky layer is adaptable to small structural settlements;
- Cost - rock material can, sometimes, be extracted close to the construction sites, lowering the transport cost;
- Visual impact - rockarmour layer integrates better into the surrounding environment than artificial units.

2.2.2 Concrete

Concrete armour units are used when suitable sized stone are not available or the design conditions require a level of stability that is not cost-effective to achieve by means of rock armour.

The widespread use of concrete in conventional construction assures a nearby source of cement and that suitable sand and aggregates are usually available. Therefore, the material cost does not vary greatly with the construction site location.

In the last fifty years, the search for better solutions to the armour layer of coastal structures lead to the development a significant amount of concrete armour units. An overview of the most important units is shown in table 2.1.

The most commonly used types of concrete armour units are (CIRIA *et al.*, 2007):

- Cubic-type blocks used in a double layer;
- Cubic-type blocks used in a single layer;
- Interlocking-type units used in a double layer;

Table 2.1: Concrete armour units (CIRIA *et al.*, 2007)

Armour unit	Country	Year	Armour unit	Country	Year
Cube	-	-	Antifer Cube	France	1973
Tetrapod	France	1050	Seabee	Australia	1978
Tribar	USA	1958	Accropode®	France	1980
Modified Cube	USA	1959	Shed	UK	1982
Stabit	UK	1961	Haro	Belgium	1984
Akmon	NL	1962	Diode	UK	1984
Tripod	N	1962	Hollow Cube	Germany	1991
Dolos	RSA	1963	Core-loc	USA	1996
Cob	UK	1969	Xblock	NL	2003

Accordingly to Burchart and Hughes (2011a), steel reinforcement as been used in the past, but the cost of reinforcement is high and usually concrete armour units are unreinforced. Therefore, are vulnerable to tension breakage above certain size and any movement of placed unit could cause breakage. The stability of armour layers is very affected if the armour units disintegrate because this reduces the stabilizing gravitational force and decreases interlocking effects. Moreover, broken armour unit pieces can be thrown around by wave action and accelerate breakage in nearby units. In order to prevent breakage it is necessary to ensure the structural integrity of the armour units. This is particularly important in the more slender units, for being the most vulnerable to cracking and breaking due the limited cross-sectional areas that give rise to relatively large tensile stresses. For this reason, usually the design criteria for this units is based on the assumption of marginal displacement of the units after placement. In order to mitigate the possibility of breakage, special precautions should be taken:

- Use of high quality concrete in casting concrete armour units;
- Proper vibration of the concrete to ensure the removal of all voids;
- Proper curing of the units before placement in the armour layer;
- Special attention to the formation of thermal cracks due to rapid curing;
- Use special equipment to handle, transport and place the armour units.

An important note is that some of the mentioned units could be patented, which implies the payment of royalties to the patent holder in order to use the unit. In small projects, this cost could be significant in the overall cost of the project. The status of patents should be reviewed before they are considered. As an example, the Tetrapod, Quadripod and Tribar are patented units, however, the patent has expired in the U.S.A., but the patent on this units may still be in force in other countries (USACE, 1984).

Burchart and Hughes (2011c) gives the fowling classification based on the structural strength of the unit:

- Massive or blocky (*e.g.* cubes, Antifer);
- Bulky (*e.g.* Accropode®, Core-loc, Haro, seabee);
- Slender (*e.g.* Tetrapod, Dolos);
- Multi-holes cubes (*e.g.* Shed, Cob).

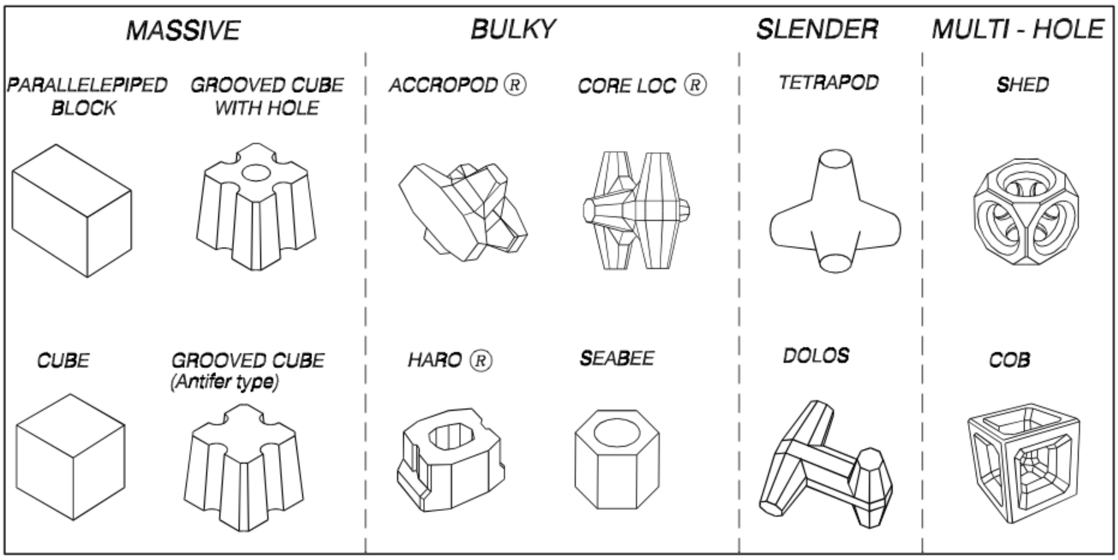


Figure 2.6: Examples of concrete armour units (Burchart and Hughes, 2011c)

2.3 Stability of Coastal Structures

The classification and direct comparison of structures as to the stability is possible through the parameter *Stability Number* (N_s), defined in equation 2.1.

$$N_s = \frac{H}{\Delta D} \tag{2.1}$$

Where:

- H - Wave height;
- Δ - Relative mass density;
- D - Characteristic diameter of the structure, armour unit, stone, gravel or sand.

For the same wave height, small values of N_s represent structures with large armour units and large values of N_s represent structures with small armour units (e.g. dynamic slopes consisting of rough gravel).

The classification of coastal structures by the stability number is given in Van der Meer (1988a) as follows:

- $N_s < 1$ - **Caissons or seawalls** - fig 2.7(a)
  No damage is allowed. The D can be the height or width of the structure.

- $N_s = 1$ to 4 - **Stable breakwaters** - fig 2.7(b)
  Only little damage is allowed under severe design conditions. Slopes are covered with heavy artificial armour units or rock. D is a characteristic diameter of the unit.

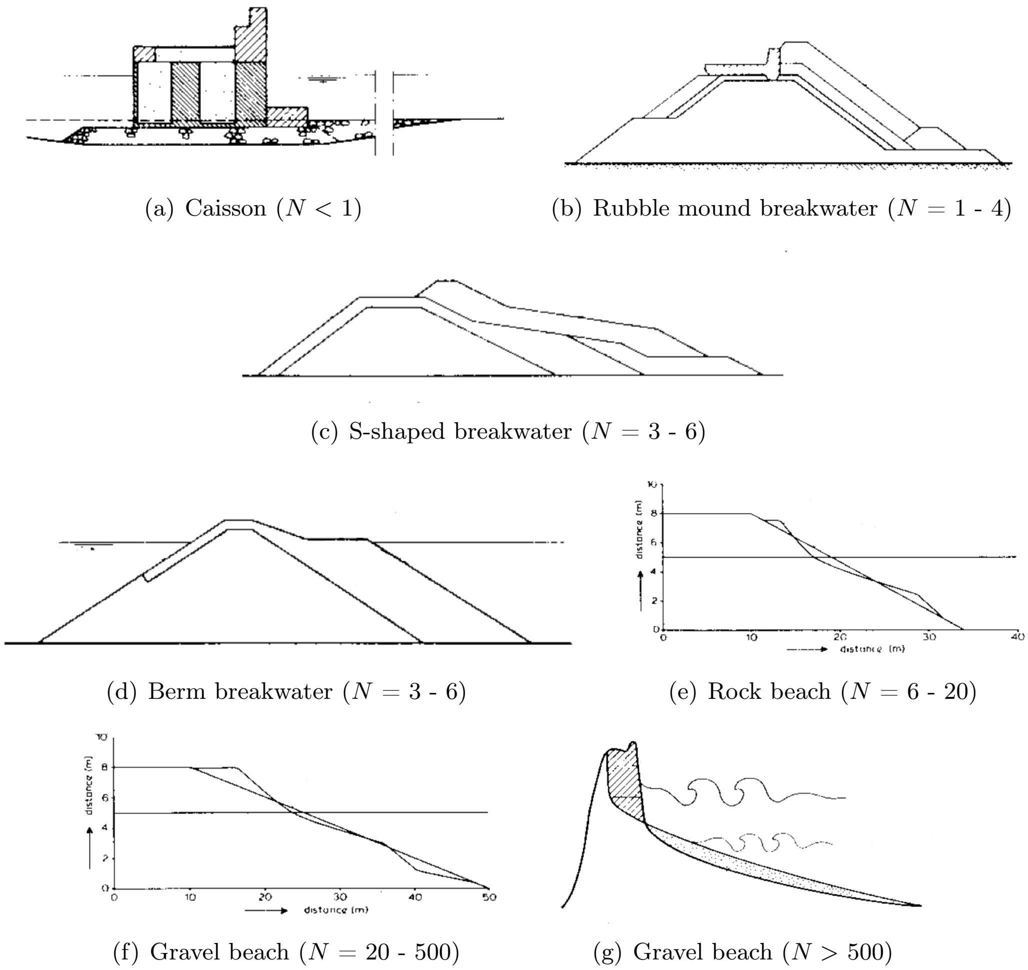
- $N_s = 3$ to 6 - **Dynamic/reshaping breakwaters** - fig 2.7(c) and fig 2.7(d)
  Reshaping breakwaters are designed with armour units in a steep slope and a horizontal berm just above the water level. The first storms develop a more gentle profile which does not change later on. The profile changes to be expected are important.

- $N_s = 6$ to 20 - **Rock slopes/beaches** - fig 2.7(e)
  The diameter of the rock is relatively small and displacement of material is allowed during severe wave attack. The design objective is the different profiles developed under wave boundary conditions.

- $N_s = 20$ to 500 - **Gravel beaches** - fig 2.7(f)
  The grain diameter in the gravel varies between 4 and 10 centimeters. The design

objective is the different profiles developed under wave boundary conditions. The profile will change continuously under different wave conditions and tide levels.

- $N_s > 500$ - **Sand beaches** - fig 2.7(g)
  Material with very small diameters can withstand severe wave attack. Dune erosion and profile development during storms are the main design parameters.



(a) Caisson ($N < 1$)

(b) Rubble mound breakwater (N = 1 - 4)

(c) S-shaped breakwater (N = 3 - 6)

(d) Berm breakwater (N = 3 - 6)

(e) Rock beach (N = 6 - 20)

(f) Gravel beach (N = 20 - 500)

(g) Gravel beach ($N > 500$)

Figure 2.7: Type of structure as a function of N (Van der Meer, 1988a)

Most breakwaters are designed in a way that only little or no damage is allowed in the design criteria. This leads to large concrete structures or heavy rock and concrete units for the armour layer. A more economic solution can be a structure with smaller elements, where profile development is allowed in order to reach a stable profile (Van der Meer, 1988a). Considering this, coastal structures exposed to direct wave attack can be roughly classified as static stable structures and dynamic stable structures, depending on the behavior under design conditions.

In statically stable structure, minor or no damage is allowed for the design conditions and damage is defined as the displacement of armour units. Statically stable structures are dependent on the individual mass and interlock capabilities of the armour units to withstand the wave forces. The design of such structures is based on an optimum solution between allowable damage and costs of construction and maintenance. Traditionally designed breakwaters belong to the group of statically stable structures. Statically stable structures have stability numbers (N_s) in the range of 1 to 4 (CIRIA *et al.*, 2007).

CIRIA *et al.* (2007) defines dynamically stable structures as structures that are allowed to be reshaped by wave attack, resulting in a development of their profile. When the profile has reached its equilibrium, its shape is maintained even if material around the still water level is continuously moving during each run-up and run-down of the waves. Dynamically stable structures are characterized by a design profile, and can roughly be classified by $N_s > 6$. In this document, dynamically stable structures will not be discussed further.

Structures with stability numbers (N_s) ranging from 4 to 6 are rock slopes or gravel beaches than can be divided into statically and dynamically stable structures where the stability of individual stones is concerned in the case of statically stable structures and the transport capacity along the slope is concerned in the case of dynamically stable structures. The range where statically stability passes to into dynamically stability is the most difficult area to describe as both the stability of individual stones and the transport capacity along the slope must be taken into account (Van der Meer, 1988a).

Chapter 3

Design of coastal structures

In this chapter, a review of the design tools for a coastal structure is presented, including the design manuals, evaluation methods and empiric formulas.

Coastal structures are, by default, very expensive to design and build. The design process is long and still based on the designers experience, that work with sometimes high levels of uncertainty.

3.1 Reference manuals for coastal structures design

In this section, three of the main international reference guides for coastal structures, and more specifically breakwaters, are referred.

Shore Protection Manual and Coastal Engineering Manual

First published by the US Army Corps of Engineers in 1974, and update in 1984, the Shore Protection Manual has been the main basis for practice in the field of coastal engineering. Approximately 30.000 copies have been sold through the U.S. Government Printing Office. Translations into other languages, including Chinese and Catalonian (Spanish), further demonstrate the SPM's role as an international standard guidance for professional coastal engineers (Pope, 1998). The advent of numerical models, reliable field instrumentation techniques, and improved understandings of the physical relationships which influence coastal processes lead to more sophisticated approaches in shore protection design in the later 1980s and 90s. In order to reflect the most up-to-date technology and knowledge of coastal processes and engineering, the Coastal and Hydraulics Laboratory (CHL) initiated preparation of the Coastal Engineering Manual (CEM) in the mid-1990's. This manual aims to gather all the relevant information in state-of-the-art coastal engineering and to provide appropriate guidance for application of techniques and methods to the solution of most coastal engineering problems (Pope, 1998).

BS6349

The BS6349 is a code of good practice regarding Maritime Structures, published by the British Standard Institution. For the theme discussed in this paper, the focus is in "BS6349- 7: Guide to the design and construction of breakwaters", that provides guidance throughout the entire process of designing a breakwater. Published in 1991, minor corrections were carried out in 2010. In its drafting, it has been assumed that the executions of its provisions is entrusted to appropriately qualified and experienced people, and that engineering judgment should be applied to determine when the recommendations of the code should be followed and when they should not (BSI, 2000).

CIRIA (The Rock Manual)

The Construction Industry Research and Information Association (CIRIA) first produced the "Manual on the use of rock in coastal and shoreline engineering" in 1991. Since this publication, significant improvements were made in the understanding of rock behavior and on hydraulic engineering practices. Consequently, the manual was updated to the present version, published in 2007 and is intended to be an extensive summary of good practice on the use of rock in engineering works for rivers, coasts and seas. It incorporates all the significant advances in knowledge that have occurred over the past 15 years (CIRIA *et al.*, 2007).

3.2 Stability evaluation methods

The research in coastal engineering is normally based on numerical and physical modeling processes. The capacity of processing large amounts of data as lead to the an increase use of numerical modeling, especially in areas where physical modeling is not or hardly possible. Expensive physical models were replaced by cheaper and faster numerical models. Wave penetration into harbours is an example (Van der Meer, 1988a).

Contrary to other domains in coastal engineering, stability of coastal structures is mostly studied by preforming test on physical models. One of the reasons for this is the relative simplicity of modeling the structures and its loads in small scale models based on Froude's law. The other main reason is that the stability is expressed by a large number of governing variables and only part of them can be described by theoretical approaches. The complex flow of waves attacking the structure makes it impossible to calculate the flow forces impacting the structure. Moreover, the complex shape of units together with their random placement makes calculation of the reaction forces between adjacent armour units impossible. Consequently, deterministic calculations of the instantaneous armour unit stability conditions cannot be performed, which is why stability formulas are based on hydraulic model tests (Burchart and Hughes, 2011a). However, the use of physical models suffers from the possibility of model and scale effects. Scale effects occur when physical properties of the structure or its individual components cannot be scaled properly. Large scale models might overcome this problem. Model effects occur from the incorrect definition of test conditions and problems associated with the use of a basin such as parasitic reflection and resonance from model boundaries. Considering this, Pita (1985) argues that the systematic observation of the behavior of existing structures helps to predict the behavior of future structures and contributes to the design process of similar structures.

Many empirical methods for the prediction of the size of armourstone required for stability under wave attack have been proposed in the last decades. Research work by Hudson (1953, cited by CIRIA *et al.*, 2007), Hudson (1959, cited by CIRIA *et al.*, 2007) and Van der Meer (1988a) has resulted in the most widely used empirical formulas in the coastal engineering world (CIRIA *et al.*, 2007). Hudson's formula is at present the most widely used and despite its limitations has the advantages of relative simplicity and the largest accumulation of experience in its use, worldwide.

As a result of more recent research in Holland into static and dynamic stability of rubble mound revetments and breakwaters, Van der Meer has proposed alternative formulas. These stability formulas are more complex than the Hudson formula, but take account some variables, which are not included on it, such as, wave period, wave breaking conditions, duration of storm and permeability of the core. The main problem encountered when using Van der Meer's formulas is the definition of the porosity parameter for the structure (Powell, 1986, cited by CIRIA *et al.*, 2007).

The choosing of which formula to use should be made considering the availability of data for the parameters required by each formula. If all input parameters are available (being sufficiently accurate) and more than one formula is considered to be valid for the desired application (some formulas are not considered valid in specific situations), it is advised to perform a sensitivity analysis on the choice of the stability formula (CIRIA *et al.*, 2007).

No stability formulas contain explicitly all the parameters affecting the stability of the structure. This together with the stochastic nature of wave load and armour response introduces uncertainty in the ability of any of the formulas to cover all the hydrodynamic effects on the structure. It is recommended that these formulas should be used only for preliminary design proposes of rubble mound breakwaters, and require confirmation and optimization with physical model tests.

3.3 Hudson formula

Based on extensive small-scale model testing and some preliminary verification by large-scale model testing, Hudson presented a formula that relates the median weight of the armour unit, with the wave height at the toe of the structure.

The presented formula was (USACE, 1984):

$$W = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot\theta} \tag{3.1}$$

Where:

- W - unit weight in Newtons of an individual armour unit in the primary cover layer. When the cover layer is two quarrystones in thickness, the stones comprising the primary cover layer can range from about $0.75W$ to $1.25W$, with about 50% of the individual stones weighing more than W;
- γ_r - specific weight of the unit's material (N/m^3);
- H - design wave height (m);
- S_r - specific gravity of unit, relative to the water at the structure $(S_r = \gamma_r/\gamma_w)$;
- γ_w - specific weight of water (N/m^3);
  - Fresh water = 9.800 (N/m^3);
  - Seawater = 10.047 (N/m^3);
- θ - structure angle measured from horizontal (degrees);
- K_D - dimensionless stability coefficient that varies with the hydrodynamic conditions and structural behavior of the armour units. This coefficient is looked in more detail later in chapter 4.

Although no tests with random waves had been conducted, the initial suggestion in USACE (1977) is to use the *significant wave height* (H_S), defined by the average of the highest one third of the waves in a time series (Van der Meer, 1988a), as the design wave height. This recommendation was revised in USACE (1984) as to use the *average of highest 10% of all waves* ($H_{1/10}$), that equals to $1.27H_S$, as the design wave height. This revision leads to a significant increase in unit weight (Burchart and Hughes, 2011a).

The fact that the structure is designed for a certain return period, does not mean that it will not sustain any damage during its life time. A probabilistic approach to the design should be considered, otherwise it would result in a over-costly structure. This probabilistic design approach acts on the definition of the project wave.

The K_D values suggested for design are presented later in this paper, and correspond to a 0-5% damage. This is the acceptable damage for design purposes and is generally referred as the "no damage" condition. Higher damage percentages have been determined as a function of the wave height for several of the armour unit shapes by Jackson (1968). Table 3.1 shows $H/H_{D=0}$ as a function of the damage percentage. H is the significant design wave height corresponding to damage $D_{\%}$ and $H_{D=0}$ is the design wave height corresponding to the no damage condition (0-5% damage). Table 3.1 is an example and values are only valid for breakwater trunk, randomly placed armour units in two layers, non-breaking waves and minor overtopping conditions. For different conditions, Jackson (1968) should be consulted to determine the correct relation.

Table 3.1: Relation between relative wave height and damage level (USACE, 1984)

Unit	Damage in Percent ($D_{\%}$)						
	0 to 5	5 to 10	10 to 15	15 to 20	20 to 30	30 to 40	40 to 50
Quarrystone (smooth)	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarrystone (rough)	1.00	1.08	1.19	1.27	1.37	1.47	1.56[a]
Tetrapods & Quadripods	1.00	1.09	1.17[b]	1.24[b]	1.32[b]	1.41[b]	1.50[b]
Tribar	1.00	1.11	1.25[b]	1.36[b]	1.50[b]	1.59[b]	1.64[a b]
Dolos	1.00	1.10	1.14[a b]	1.17[a b]	1.20[b]	1.24[a b]	1.27[a b]

[a] Values interpolated or extrapolated;
[b] Waves exceeding the design wave height conditions by more than 10% may result in considerably more damage than the values in the table.

The values in table 3.1, together with statistical data concerning the frequency of occurrence of waves of different height, can be used to determine the annual cost of maintenance as a function of the damage to the structure. Knowledge of maintenance costs can be used to choose a design wave yielding the optimum combination of first and maintenance costs (Figure 3.1).

The main advantage of the Hudson formula is its simplicity and the wide range of armour units and configurations for which K_D values have been derived. This formula has, however, limitations:

- The use of regular waves only;

- No account of the wave period and the storm duration;

- No description of the damage level;
- The use for non-overtopped and permeable structures only.

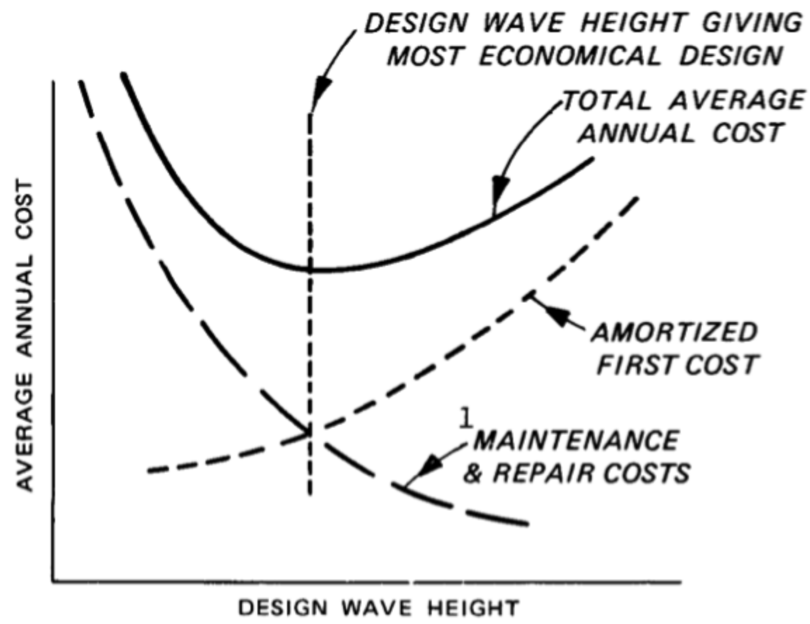


Figure 3.1: Optimum combination of first and maintenance costs, adapted from Lima (2011)

3.4 Van der Meer formulas

Considering the limitations of Hudson's equation, described in section 3.3, various approaches were presented by numerous authors, among which is the Van der Meer formula, presented in Van der Meer (1988a). This formula, valid only for deep water conditions, considers variables like a clearly defined damage level (S_d), permeability of the core and inner layers (P), duration of the storms characterized by the number of waves (N), wave period (T) and the different types of waves attacking the structure (plunging or surging waves).

3.4.1 Deep water conditions

Deep water is for the purpose of the validity of these formulas defined as: the water depth at the toe of the structure is larger than three times the significant wave height at the toe ($h > 3H_{S,toe}$) (CIRIA *et al.*, 2007).

Van der Meer's formula presented in Van der Meer (1988a) is only valid to predict the stability of armourstone slopes with crests above maximum run-up level (CIRIA *et al.*, 2007). The formulas make use of a distinction between plunging waves and surging waves. This distinction is made by comparing the mean surf similarity parameter (ξ_m), calculated with equation 3.4, and a critical surf similarity parameter (ξ_{cr}), calculated using equation 3.5. If $\xi_m < \xi_{cr}$, waves are plunging and equation 3.2 applies, and if $\xi_m \geq \xi_{cr}$, waves are surging and equation 3.3 applies (CIRIA *et al.*, 2007). Irrespective of this, for slopes more gentle than 1:5 ($\cot\alpha \geq 4$) only equation 3.2 applies.

Table 3.2 shows the range of validity of the various parameters used in Equations 3.3 and 3.2.

Van der Meer formula for plunging waves (Burchart and Hughes, 2011a):

$$\frac{H_S}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \tag{3.2}$$

Van der Meer formula for surging waves (Burchart and Hughes, 2011a):

$$\frac{H_S}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_m^{P} \tag{3.3}$$

Where:

- N - number of incident waves, which depends on the duration of the wave conditions;
- H_s - significant wave height at the structure site;
- ξ_m - surf similarity parameter calculated using equation 3.4;
- α - slope angle;
- Δ - buoyant density $\left(\frac{\rho_r}{\rho_w} - 1\right)$;
- ρ_r - mass density of rock;
- ρ_w - mass density of water;
- P - notional permeability of the structure; the value of this parameter range from 0.1, for a relatively impermeable core, up to 0.6 for a virtually homogeneous rock structure (see figure 4.7). The choice of P to be used in design depends on judgment and it is recommended that the permeability be underestimated rather than overestimated (BSI, 1991);
- D_{n50} - median nominal size of the armourstone;

- S_d - Considered damage level (see section 4.5).

The mean surf parameter is calculated by (CIRIA *et al.*, 2007):

$$\xi_m = \frac{\tan\alpha}{\sqrt{\frac{2\pi}{g} \times \frac{H_S}{T_m^2}}} \tag{3.4}$$

Where:

- T_m represents the mean wave period;
- g represents the gravity acceleration, g = $9.81 m/s^2$.

The critical surf parameter is calculated by (CIRIA *et al.*, 2007):

$$\xi_{cr} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} \tag{3.5}$$

Table 3.2: Range of validity of parameters in Van der Meer formulas for deep water conditions (CIRIA *et al.*, 2007)

Parameter	Symbol	Range
Slope angle	$\tan\alpha$	1:6 - 1:1.5
Number of waves	N	< 7500
Fictitious wave steepness based on T_m	s_{om}	0.01 - 0.06
Surf similarity parameter using Tm	ξ_m	0.7 - 7
Relative buoyant density of armourstone	Δ	1 - 2.1[a]
Relative water depth at toe	$h/H_{S,toe}$	> 3
Notional permeability parameter	P	0.1 - 0.6
Armourstone gradation	D_{n85}/D_{n15}	< 2.5
Damage-storm duration ratio	$S_d/\sqrt{N}$	< 0.9
Stability number	N_s	1.0 - 4.0
Damage level parameter	S_d	1.0 - 20.0

[a] For higher values of the relative buoyant density (up to $\Delta \cong 3.5$) the validity of the stability formulas is restricted to structures with front slopes with $\cot\alpha \geq 2$ (Helgason and Burchart, 2005, cited by CIRIA *et al.*, 2007);

3.4.2 Shallow water conditions

The definition of shallow water is relevant to limit the field of application on the Van der Meer formula (equations 3.2 and 3.3), developed for use in deep water only. Accordingly to CIRIA *et al.* (2007), some researchers define the transition from deep to shallow water around the water depth equal to *three times the significant wave height at the toe of the structure* ($H_{s,toe}$). Other researchers who studied conditions with very shallow foreshores, have defined very shallow water (where a considerable amount of wave breaking occurs) as the condition at which $H_{S,toe} < 70\%$ of the *deep water wave height* (H_{so}) (Van Gent, 2003a, cited by CIRIA *et al.*, 2007).

In shallow water, combinations of individual wave height, period, and bottom depth can result in individual waves or groups of waves significantly larger than H_s. The shape of the waves becomes more peaked and wave breaking becomes more common. Due to these facts, the distribution of the wave heights deviates from the Rayleigh distribution (CIRIA *et al.*, 2007). In order to take into account the effect of the changed wave distribution, Van der Meer (1988a) states that the stability of the armour layer in these depth-limited conditions is better described by using the *top 2 per cent wave height* ($H_{2\%}$), than by the significant wave height (H_s). Considering the ratio defined in table 4.7 ($H_{2\%}/H_s = 1.4$), the Van der Meer formulas for deep water conditions (equations 3.2 and 3.3) can be rewritten to determine the stability of armour layer for structures in shallow water conditions. Using the same process as for calculating the armour stability in deep water conditions, the stability formulas for shallow water conditions is presented in Van der Meer (1988a) as equation 3.6 for plunging waves and equation 3.7 for surging waves.

Van der Meer formula for plunging waves (Van der Meer, 1988a):

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7 P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{3.6}$$

Van der Meer formula for surging waves (Van der Meer, 1988a):

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4 P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \tag{3.7}$$

Based on analysis of the stability of rock-armoured slopes focused on shallow water conditions, Van Gent *et al.* (2003a, cited by CIRIA *et al.*, 2007) proposed a modification of the formula presented in Van der Meer (1988a) for shallow water conditions. One of the modifications was to use the *spectral wave period* ($T_{m-1.0}$) instead of the *mean wave period* (T_m) in order to take the influence of the shape of the wave energy spectra into account. Considering a fix relation between $T_{m-1.0}$ and T_m, the coefficients to apply to the formula were obtained using tests on physical models. This resulted in the modified stability formulas presented as equations 3.8 and 3.9.

Table 3.3 shows the range of validity of the various parameters used in equations 3.8 and 3.9.

Modified Van der Meer equation for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 8.4P^{0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\left(\frac{H_s}{H_{2\%}}\right)(\xi_{s-1.0})^{-0.5} \tag{3.8}$$

Modified Van der Meer equation for surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.3P^{-0.13}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\left(\frac{H_s}{H_{2\%}}\right)\sqrt{\cot\alpha}\,(\xi_{s-1.0})^{P} \tag{3.9}$$

Where:

- $H_{2\%}$ - Wave height exceeded by 2% of the incident waves at the toe (m);
- $\xi_{s-1.0}$ represents the surf similarity parameter, calculated using equation 3.4 with $T_{m-1.0}$;

Table 3.3: Range of validity of parameters in modified Van der Meer formulas for shallow water conditions (CIRIA *et al.*, 2007)

Parameter	Symbol	Range
Slope angle	$\tan\alpha$	1:4 - 1:2
Number of waves	N	< 3000
Fictitious wave steepness based on T_m	s_{om}	0.01 - 0.06
Surf similarity parameter using T_m	ξ_m	1 - 5
Surf similarity parameter using $T_{m-1.0}$	$\xi_{s-1.0}$	1.3 - 6.5
Wave height ratio	$H_{2\%}/H_s$	1.2 - 1.4
Deep-water wave height over water depth at toe	H_{so}/h	0.25-1.5
Notional permeability parameter	P	0.1 - 0.6
Core material - armour ratio	$S_d/\sqrt{N}$	0 - 0.3
Stability number	N_s	1.0 - 4.0
Damage level parameter	S_d	1.0 - 20.0

From the same data as the modified Van der Meer equations, Van Gent *et al.* (2003a, cited by CIRIA *et al.*, 2007) derived a more simple formula based on the principle that the wave period influence decreases significantly when very shallow conditions are considered (CIRIA *et al.*, 2007). The derived formula is presented as equation 3.10 (where D_{n50} is the median nominal size of the armourstone and $D_{n50,core}$ is median nominal size of the core material) and leads to more or less the same accuracy as equations 3.8 and 3.9. This equation is intended specially when no accurate information concerning the wave period and wave height ratio are available.

$$\frac{H_s}{\Delta D_{n50}} = 1.75\sqrt{\cot\alpha}\left(1+\frac{D_{n50,core}}{D_{n50}}\right)^{2/3}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \tag{3.10}$$

3.4.3 Concrete armour layer

Similar test as presented in Van der Meer (1988a), for rock armour layers, were performed in Delft University hydraulics laboratory, on breakwaters armoured with Cubes, Tetrapods and Accropode®.

From the test results, formulas for the prediction of the stability of layers armoured with such units, was defined. These formulas are presented in Van der Meer (1988b) for Cubes (equation 3.11), Tetrapods (equation 3.12) and Accropode® (equations 3.13 and 3.14).

Stability formula for Cubes:

$$\frac{H_S}{\Delta D_{n50}} = \left(6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \tag{3.11}$$

Stability formula for Tetrapods:

$$\frac{H_S}{\Delta D_{n50}} = \left(3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} \tag{3.12}$$

Where:

- N_{od} - Relative damage level (see section 4.5 of this document)
- s_{om} - Slope of the fictitious wave ($S_{om} = \frac{H_s}{L_{om}}$)
- L_{om} - Length of the offshore wave (m)

Accropodes® are placed in a one layer system. From the test result analysis on Van der Meer (1988b), the author states that the stability for *start of damage* ($N_{od} = 0$) is very high when compared to Cubes and Tetrapods. However, *severe damage* or *failure* ($N_{od} > 0.5$) are very close to start of damage. This behavior is consistent with one-layer armour solutions. The stability formulas for Accropodes® are presented in equations 3.13 and 3.14.

Stability formula for Accropode® (start of damage):

$$\frac{H_S}{\Delta D_{n50}} = 3.7 \tag{3.13}$$

Stability formula for Accropode® (failure):

$$\frac{H_S}{\Delta D_{n50}} = 4.1 \tag{3.14}$$

Formulas for Cubes and Tetrapods do not distinguish between surging and plunging waves, as the formulas for armourstone do. This probably due to the steep slope considered in the tests. However, Jong) (1996, cited by Van der Meer, 1999) performed more tests on tetrapods and found a transition from surging to plunging waves similar to the one found in rock and derived a formula for plunging waves (equation 3.15). (Van der Meer, 1999) states that for the assessment of the stability of Tetrapods, equation 3.12 should be used in the case of surging waves, and equation 3.15 should be used for plunging waves.

Stability formula for Tetrapods (plunging waves):

$$\frac{H_S}{\Delta D_{n50}} = \left(8.6\frac{N_{od}^{0.5}}{N^{0.25}} + 3.94\right) s_{om}^{-0.2} \tag{3.15}$$

3.5 Summary of the presented stability formulas

As presented in this document, various stability formulas are available to predict the stability or armour layers in breakwaters.

The choice of the stability formula to consider should depend first on the validity of the formula for the desired conditions and considered units (e.g. some formulas are not valid for concrete armour units or shallow water conditions). Secondly the designer should ascertain if all the necessary input parameters are available and are sufficient accurate for each of the considered formulas. If all input parameters are available and more than one formula is considered to be valid for the desired application, CIRIA *et al.* (2007) advises the designer to perform a sensitivity analysis on the choice of the stability formula.

Table 3.4 gives an overview of the validity of the formulas considered in this document and the input parameter dependency for each of them. The correlation between the formulas presented in table 3.4 and the formulas presented earlier in this document is as follows:

- Hudson is given by equation 3.1;
- Van der Meer for deep water is given by equations: 3.2 and 3.3 for plunging and surging conditions, respectively;
- Modified Van der Meer is given by equations 3.8 and 3.9 for plunging and surging conditions, respectively;
- Van Gent *et al.* is given by equation 3.10.

Table 3.4: Overview of fields of application of different stability formulas for rock armoured slopes (CIRIA *et al.*, 2007)

Criteria	Hudson	Van der Meer deep water	Modified Van der Meer	Van gent *et al*
Deep water conditions[a]	Yes	Yes	No	No
Very deep water conditions[b]	No	No	Yes	Yes
Permeable core	Yes[c]	Yes	Yes	Yes
Impermeable core	Yes[d]	Yes	Yes	No
Design experience	Yes	Yes	Limited	No
Number of waves dependent	No	Yes	Yes	Yes
Wave period dependent	No	Yes[e]	Yes[f]	Yes
Wave height H2% dependent	No	No	Yes	No
Permeability dependent	No	Yes	Yes	No
Core D_{n50} dependent	No	No	No	Yes

[a] $h > 3H_{s,toe}$
[b] $H_{s,toe} < 70\% of H_{so}$
[c] For $K_D = 4$
[d] For $K_D = 1$
[e] T_m
[f] $T_{m-1.0}$

Chapter 4

Hudson stability coefficient

The dimensionless stability coefficient (K_D), used in Hudson Formula, accounts for all parameters that determine the hydraulic performance and structural response of a breakwater, other than the ones specified in the formula. Numerous researchers with a view to establishing values of K_D, performed a wide range of laboratory tests for various conditions on some of the variables. The main variables affecting K_D are discussed in this chapter.

4.1 Typical stability coeficient

The tables presented in this chapter (from table 4.1 to table 4.4), show some typical K_D values referred in the literature, for different situations, depending on the type of armour unit, manner of placement, packing density and number of layers (n).

In addition to the armour units present in the USACE (1977) and USACE (1984), the K_D values for some of the most used concrete armour units are presented in table 4.3.

Table 4.1: Comparison between K_D values in USACE (1977) and USACE (1984) for structure trunk

Armour Units	n	Placement	Structure Trunk[a] SPM 1977 Breaking	SPM 1977 Nonbreaking	SPM 1984 Breaking	SPM 1984 Nonbreaking
Quarrystone						
Smooth Rounded	2	Random	2.1	2.4	1.2[b]	2.4
Smooth Rounded	>3	Random	2.8	3.2	1.6[b]	3.2[b]
Rough Angular	1[d]	Random	-	2.9	-	2.9[b]
Rough Angular	2	Random	3.5	4.0	2.0	4.0
Rough Angular	>3	Random	3.9	4.5	2.2[b]	4.5[b]
Rough Angular	2	Special[e]	4.8	5.5	5.8	7.0
Parallelepiped[f]	2	Special	-	-	7.0 - 20.0	8.5 - 24.0[b]
Concrete armour units						
Tetrapod and Quadripod	2	Random	7.2	8.3	7.0	8.0
Tribar	2	Random	9.0	10.4	9.0[b]	10.0
Dolos	2	Random	22.0[c]	25.0[c]	15.8[c]	31.8[c]
Modified Cube	2	Random	6.8	7.8	6.5[b]	7.5
Hexapod	2	Random	8.2	9.5	8.0[b]	9.5
Toskane	2	Random	-	-	11.0[b]	22.0
Tribar	1	Uniform	12.0	15.0	12.0[b]	15.0

[a] Applicable to slopes ranging from 1:1.5 to 1:5;
[b] These K_D values are unsupported by test results and are only provided for preliminary design purposes;
[c] Refers to no-damage criteria (<5% displacement, rocking, etc.); if no rocking (<2%) is desired, reduce KD by 50 percent (Zwamborn and van Niekerk, 1982, cited by USACE, 1977);
[d] The use of a single layer of quarrystone armour units is not recommended for structures subject to breaking waves and only under special conditions for structures subject to nonbreaking waves. When used, the stone should be carefully placed;
[e] Special placement with long axis of stone placed perpendicular to structure face;
[f] Parallelepiped-shaped stone: long slab-like stone with the long dimension about 3 times the shortest dimension (Markle and Davidson, 1979, cited by USACE, 1977).

Table 4.2: Comparison between K_D values in USACE (1977) and USACE (1984) for structure head

Armour Units	n	Placement	Structure Head SPM 1977 Breaking	SPM 1977 Non	SPM 1984 Breaking	SPM 1984 Non	Slope $\cot\theta$
Quarrystone							
Smooth Rounded	2	Random	1.7	1.9	1.1^a	1.9	1.5 to 3.0
Smooth Rounded	>3	Random	2.1	2.3	1.4^a	2.3^a	^{c}
Rough Angular	1^d	Random	-	2.3	-	2.3^a	^{c}
Rough Angular	2	Random	2.9	3.2	1.9^a	3.2	1.5
			2.5	2.8	1.6^a	2.8	2.0
			2.0	2.3	1.3	2.3	3.0
Rough Angular	>3	Random	3.7	4.2	2.1^a	4.2^a	^{c}
Rough Angular	2	Special[e]	3.5	4.5	5.3^a	6.4^a	^{c}
Concrete armour units							
Tetrapod and Quadripod	2	Random	5.9	6.6	5.0^a	6.0	1.5
			5.5	6.1	4.5^a	5.5	2.0
			3.7	4.1	3.5^a	4.0	3.0
Tribar	2	Random	8.3	9.0	8.3^a	9.0	1.5
			7.8	8.5	7.8^a	8.5	2.0
			7.0	7.7	6.0^a	6.5	3.0
Dolos	2	Random	15.0	16.5	8.0^a	16.0^a	2.0^b
			13.5	15.0	7.0^a	14.0^a	3.0
Modified Cube	2	Random	-	5.0	-	5.0	^{c}
Hexapod	2	Random	5.0	7.0	5.0	7.0	^{c}
Tribar	1	Uniform	7.5	9.5	7.5	9.5	^{c}

[a] These K_D values are unsupported by test results and are only provided for preliminary design purposes;
[b] Stability of dolosse on slopes steeper than 1:2 should be substantiated by site-specific model test;
[c] Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes raging from 1:1.5 to 1:3. Some armour units tested on a structure head indicate a K_D-slope dependence;
[d] The use of a single layer of quarrystone armour units is not recommended for structures subject to breaking waves and only under special conditions for structures subject to nonbreaking waves. When used, the stone should be carefully placed;
[e] Special placement with long axis of stone placed perpendicular to structure face;
[f] Cotangent of the slope angle that the structure wall makes with the horizontal.

Table 4.3: K_D values for various armour units

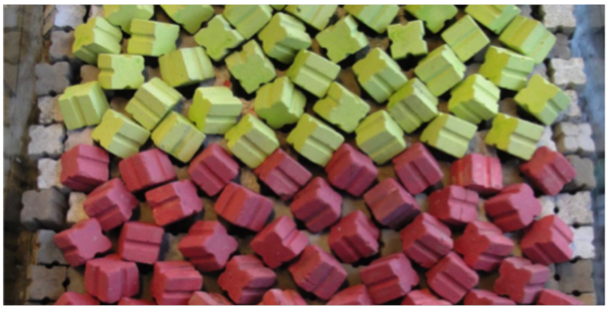
Unit	KD	Conditions	Reference
Accropode®	15.0	Non-breaking waves, trunk sections	(CLI, 2013a)
	11.5	Non-breaking waves, head sections	(CLI, 2013a)
Accropode® II	16.0	Non-breaking waves, trunk sections	(CLI, 2013a)
	12.3	Non-breaking waves, head sections	(CLI, 2013a)
CORE-LOC™	16.0	Non-breaking waves, trunk sections	(CLI, 2013a)
	13.0	Non-breaking waves, head sections	(CLI, 2013a)
ECOPODE™	16.0	Non-breaking waves, trunk sections	(CLI, 2013a)
	12.3	Non-breaking waves, head sections	(CLI, 2013a)
xblock®	16.0	Non-breaking waves, trunk sections, 4:3 slope	(DMC, 2011)
Akmon	17.0	Non-breaking waves, 1.5:1 slope	(Paape and Walther, 2011)
Sealock	10	-	(Yoo, 2010)

In the case of the Antifer block, Yalciner *et al.* (1999) presented the results of a series of tests with irregular placement of the blocks. The obtained K_D values are presented in table 4.4.

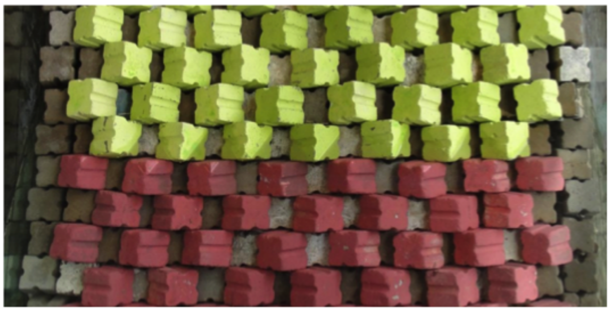
Table 4.4: K_D values for Antifer blocks

Slope	Breaking waves		Non-Breaking waves	
$\cot\theta$	Trunk	Head	Trunk	Head
1.5	4.0	3.5	5.0	4.0
2.0	5.5	4.5	7.0	5.5
2.5	6.5	5.5	8.0	6.5
3.0	7.5	6.5	9.0	7.5

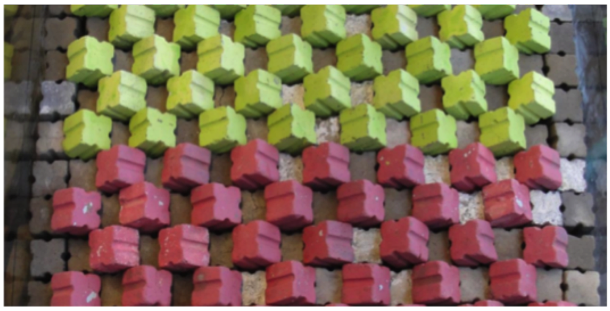
Regarding the placement of the Antifer blocks, Freitas *et al.* (2013) recommends, based on model tests, the values of K_D=2.1 for semi-irregular placement (figure 4.1(a)), K_D=5.8 for regular placement 1 (figure 4.1(b)) and K_D=4.0 for regular placement 2 (figure 4.1(c)). For the semi-irregular placement method, the considered damage level is 5%, as for both regular placement methods, the considered damage level is almost null damage, due to the fact that the armour layer cannot be easily repaired by filling up the holes, because the blocks above tend to slide down the slope. The difference between the two regular placements is the distance between the blocks, measured in the horizontal plane, the regular placement 1 has a distance of 3.39cm between two consecutive blocks, and the regular placement 2 has 4.08cm.



(a) Semi-irregular placement



(b) Regular placement 1



(c) Regular placement 2

Figure 4.1: Physical model tests to evaluate Antifer blocks placement (Frens, 2007)

Considering regular placing methods, experimental research in the wave-flume of the fluid mechanics laboratory of the Faculty of Civil Engineering and Geosciences at Delft University of Technology, the Netherlands, and presented in Frens (2007), suggests K_D values for numerous placement methods and packing densities. The range of K_D values obtained in these tests goes from 4.0 to 23.7.

4.2 Interlocking capability

The interlocking capability is the ability of a unit to work together with neighbor unit to increase the stability of the armour layer. The interlocking effect is demonstrated in figure 4.2. The interlocking effect is highly dependent on the slope of the structure. Moreover, the interlocking effect is significant only for steeper slopes (Burchart and Hughes, 2011a). Besides the slope angle, the interlock capability of a unit is affected by the units shape, roughness and placement.

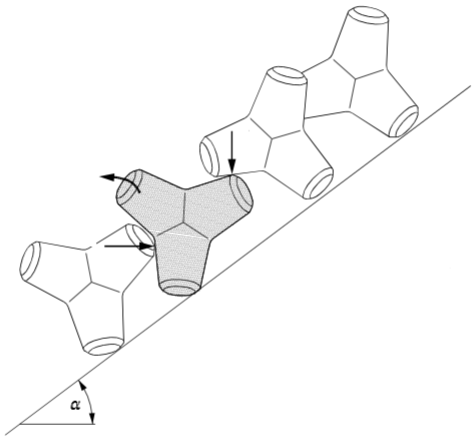


Figure 4.2: Interlocking effect (Burchart and Hughes, 2011a)

4.2.1 Unit shape

Together with the placing manner, it defines the way that the armour units interlock between themselves and work together to protect the structure against the waves. It is considered that the higher the level of interlock is, the lower is the weight necessary for each unit, as demonstrated in figure 4.3.

Angular, blocky stones are preferred for armour layers because they wedge and interlock well with adjacent stones when placed randomly, they can be placed on steeper slopes, and they provide a more porous armour layer that more effectively dissipates wave energy (Burchart and Hughes, 2011b). In the case of armourstone, tests performed by Latham *et al.* (1988, cited by CIRIA *et al.*, 2007) in slopes armoured with *round*, *standard* (*ie* rough, angular) and *tabular* revealed that *round* armourstone suffered more damage than *standard* armourstone, and *tabular* armourstone suffered less damage than *standard* armourstone. This trend is consistent with the stability coefficient values presented in section 4.1 for armourstone.

The shape of the stone is inherited from the structure of the rock mass and is not

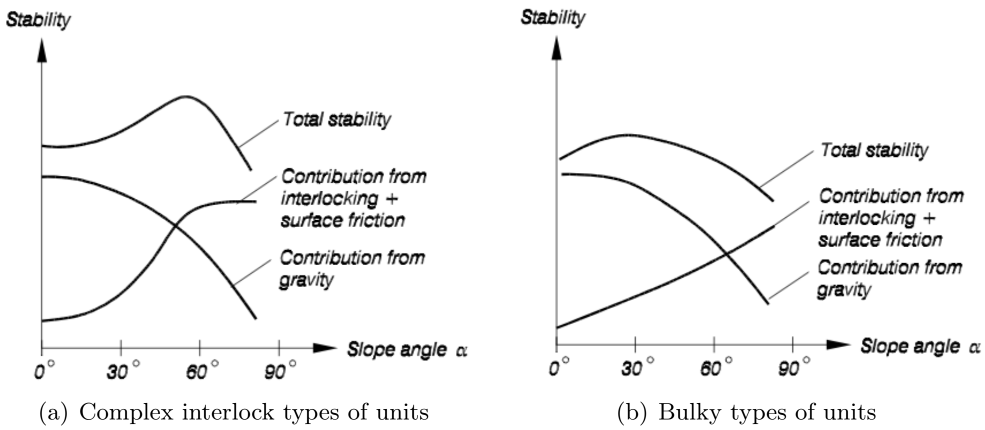


Figure 4.3: Interlock effect on complex and bulky armour units (Burchart and Hughes, 2011a)

strongly controlled by production techniques (CIRIA *et al.*, 2007). On the contrary, concrete units can be fabricated in any necessary shape within material limits. This fact, together with the search for lighter and more cost-effective units, lead to the appearance of increasingly complex units (see section 2.2.2).

4.2.2 Unit placement

Together with the shape of the unit, it defines the way that the single armour units interlock between themselves. In the design, placement can be characterized in two patterns:

- Uniform placement - Used for cut stones with approximately the same time. The units are placed in a orderly pattern that make it more difficult for units to be moved (figure 4.4(a));

- Random placement - Covers a range of placement technics from the careful placement of individual stones in a random pattern to the dumping of stones from trucks and barges (figure 4.4(b)).

Structures in which stones are carelessly placed will inevitably suffer damage at loads below design levels (Burchart and Hughes, 2011b). Therefore it is important that stones are placed following the design specifications.

4.2.3 Unit roughness

Surface roughness and sharpness of edges of armour units provide some level of interlocking between the individual units due to friction.

(a) Uniform placement

(b) Random placement

Figure 4.4: Placement of armourstone (Burchart and Hughes, 2011a)

Roughness is also important in the absorption of the incident wave energy by providing a surface with less reflection and refraction problems.

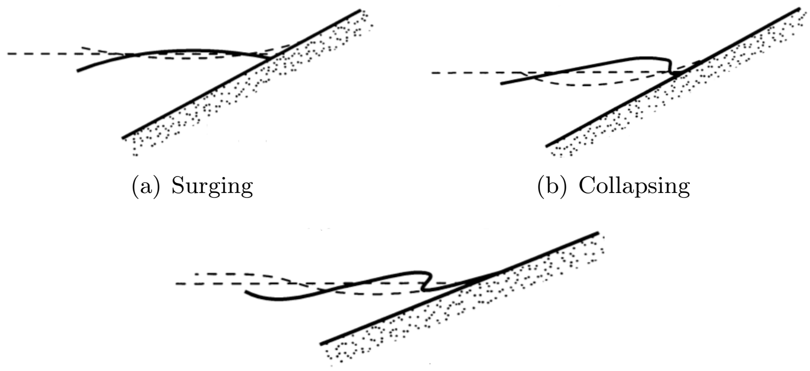
4.3 Types of wave attacking the structure

Waves attacking a structure can be defined as to their breaking characteristics as:

- Surging (figure 4.5(a))
- Collapsing (figure 4.5(b))
- Plunging (figure 4.5(c))

The breaker type is dependent on the initial wave energy, configuration of the depths and the configuration of the structure's slope.

The attack of the structure by breaking or non-breaking waves produces different loads in the armour layer. In the test results from Van der Meer (1988a), a clear difference on stability between plunging and surging waves was found. In the case of plunging waves,



(a) Surging
(b) Collapsing
(c) Plunging

Figure 4.5: Wave breaking types (BSI, 2000)

the forces caused by the fast wave run-up after breaking are the main stability concern, since the run-down forces are relatively small. The opposite is found in the case of surging waves, where the main instability is caused by the run-down forces.

Accordingly to McConnell *et al.* (2004), the impact forces of breaking waves are substantially more intense than non-breaking wave loads. This statement is supported by USACE (1977) and USACE (1984), trough the recommended stability parameter values to use in the Hudson equation. These values are, smaller for breaking waves than for non-breaking waves, which lead to heavier units under similar conditions.

Most physical model tests have been performed for non-breaking waves. The K_D values for breaking waves were estimated based on non-breaking wave tests of similar armour units (USACE, 1984). For this reason, the recommendation is that the existing values considering breaking waves should only be used for preliminary design.

4.4 Seabed slope in front of the structure

The effect of steep approach slopes on armour layer stability is still difficult to predict due to the insufficient knowledge in this matter. However, examples of damaged structures in this condition show that a safety factor should be applied to take this parameter under consideration on the preliminary design.

CIRIA *et al.* (2007) proposes, as a rule of thumb, that the stone size required for stability should be at least 10% larger than that in normal deep-water conditions. The company "Concrete Layer Innovation" includes a variation in K_D, in function of the seabed slope in front of the structure, in its on-line calculator (CLI, 2013b). The reduction of the K_D value is, accordingly to the company internet site, based on physical model results.

4.5 Damage criteria

In the process of evaluating the stability of a coastal structure, it is important to define with relative precision the amount of damage allowed to the structure as one of the design conditions.

Damage levels in an armour slope of a classical breakwater can be classified as follows (Burchart and Hughes, 2011a):

- No-damage - No unit displacement;
- Initial damage - Few units are displaced. This damage level corresponds to the *no-damage* criteria used in USACE (1977) and USACE (1984) in relation to the Hudson stability coefficient, where the *no-damage* is defined as 0 to 5% displaced units within the zone extending from the middle of the crest height to a depth below still water level equal to H_s;
- Intermediate damage - Units are displaced but without causing exposure of the under layer or filter layer to direct wave attack;
- Failure - The under layer or filter layer is exposed to direct wave attack.

One of the most common approaches to define the damage value to armourstone layers is to use a dimensionless damage parameter defined by Broderick (1983, cited by Burchart and Hughes, 2011a), presented in equation 4.1 (where A_e is the eroded area (see figure 4.6) and D_{n50} is the equivalent cube length of median stone). This parameter can be interpreted as the number of squares with side length D_{n50} which fit into the eroded area, or as the number of cubes with side length D_{n50} eroded within a strip with D_{n50} width of the armour layer. An example of the use of this parameter applied to an armourstone in a double layer is given in table 4.5.

$$S_d = \frac{A_e}{D_{n50}^2} \tag{4.1}$$

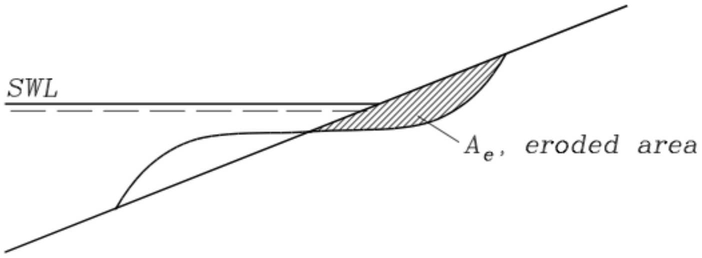


Figure 4.6: Eroded area schematization (Burchart and Hughes, 2011a)

Table 4.5: Design values of the damage parameter for armourstone in a double layer (CIRIA *et al.*, 2007)

Slope ($\cot \alpha$)	Damage Level Initial damage	Intermediate damage	Failure
1.5	2	3 - 6	8
2	2	4 - 6	8
3	2	6 - 9	12
4	3	8 - 12	17
6	3	8 - 12	17

In the case of artificial armour units, damage is often measured as the number of units displaced more than one diameter and presented in percentage. However, this approach is dependent on the slope angle and the total number of units in the armour layer. Therefore, different investigations can hardly be compared (Van der Meer, 1988b). A different definition for damage to concrete units is suggested is Van der Meer (1988b). In this case, damage can be defined as the *relative damage* (N_{od}), presented in equation 4.2, which is the actual number of units displaced related to a width (along the longitudinal axis of the structure) of one nominal diameter D_n. For cubes D_n is the side of the cube, for Tetrapods $D_n = 0.65D$, where D is the height of the unit and for Accropode® $D_n = 0.7D$ (Van der Meer, 1999). The characteristic damage level values for different concrete armour units is presented in table 4.6.

Definition of *relative damage* (Burchart and Hughes, 2011a):

$$N_{od} = \frac{\text{number of units displaced out of armour layer}}{\text{width of tested section}/D_n} \tag{4.2}$$

Table 4.6: Relative damage level for concrete armour units (Burchart and Hughes, 2011a)

Unit	Slope	Initial damage	Intermediate damage	Failure
Cube	1:1.5	0	-	2
Tetrapods	1:1.5	0	-	1.5
Accropods®	1:1.33	0	-	5

4.6 Wave height

The height of the wave attacking a structure is one of the most important variables affecting the stability of the structure. The over-evaluation of the wave height results on greater construction costs for the structure and the under-evaluation results in greater risk of damage or failure. Therefore Pinto and Neves (2003) states that the structure should be design to resist the maximum wave predicted for the structure site, but always considering the economic viability. The wave height used in stability formulas is always the wave height in front of the structure (Van der Meer, 1988a).

It is important to emphasize that the wave height is not an "exact" value, in terms that its value is associated to a certain return period, a risk level and to a probability of being exceeded. Therefore Pinto and Neves (2003) refers that a probabilistic approach is suggested to define a equivalent parameter *wave height* (H). This parameter is based on records from irregular waves and symbolizes the height of a regular wave that causes the same damage to the structure as the considered irregular waves. This process consists in sorting a a series of N waves, considered representative of a storm event, and determine the characteristic wave heights and periods. Normally, two definitions are used to represent the wave heights of a sea-state for a given duration:

- $H_{P\%}$ is the wave height that is exceeded by P per cent of the wave heights in the sea-state.

- $H_{1/Q}$ corresponds to the average height of the $1/Q$ largest wave heights in the sea-state

The more widely used values are the *significant wave height* ($H_{1/3} = H_S$) that is defined by the average of highest on third of the waves in a time series, the *average of highest 10% of all waves* ($H_{1/10}$) and the *wave height exceeded by 2% of wave heights in the record* ($H_{2\%}$).

The wave height to consider for each case is highly dependent on the implementation depth of the structure and the inclination of the bottom surface. Various authors give different criteria to define the relative depth, e.g. in a recent study, Van Gent *et al.*

(2003b, cited by Prevot *et al.*, 2012) has established a different criteria by studying the ratio of the significant wave height at the structure to that observed offshore:

- when $H_{S,toe}/H_{S,offshore} > 0.9$, the structure is in deep water;
- when $0.7 > H_{S,toe}/H_{s,offshore} > 0.9$, the conditions are said to be shallow water conditions (where shoaling occurs and there is limited wave breaking);
- when $H_{S,toe}/H_{S,offshore} < 0.7$, the conditions are said to be very shallow water conditions (where a considerable amount of wave breaking occurs).

In deep water, the individual wave heights closely follow the Rayleigh distribution and the *representative wave heights* $H_{P\%}$ and $H_{1/Q}$ can be calculated through this distribution and the *mean wave height* (H_m) (CIRIA *et al.*, 2007). The most important values are listed in table 4.7.

Table 4.7: Characteristic wave height ratios for a sea-state with a Rayleigh distribution of wave heights (CIRIA *et al.*, 2007)

Characteristic height (H)	Wave height ratios	
	H/Hm	H/Hs
Mean wave height (H_m)	1	0.626
Significant wave height (H_S)	1.597	1
Wave height ($H_{1/10}$)	2.031	1.273
Wave height ($H_{1/100}$)	2.662	1.668
Wave height ($H_{2\%}$)	2.232	1.397

Most structures are not situated in deep water. In this situation, the wave propagation is affected by phenomena like reflection, refraction, etc. Furthermore the highest waves will break on the foreshore, meaning that the wave forces attacking the structure will be reduced. This means that the assumption of a Rayleigh distribution in front of the structure cannot often be made. An alternative distribution model for wave heights in the shoaling and breaking zone can be found in Battjes and Groenendijk (2000, cited by CIRIA *et al.*, 2007), that successfully tested a composite Weibull distribution.

A more extensive study on the wave height problematic can be found in Marinho (2013). This paper addresses in detail the statistical distributions, the influence of the bottom depths and the choice of the project wave to consider for the design project.

4.7 Permeability

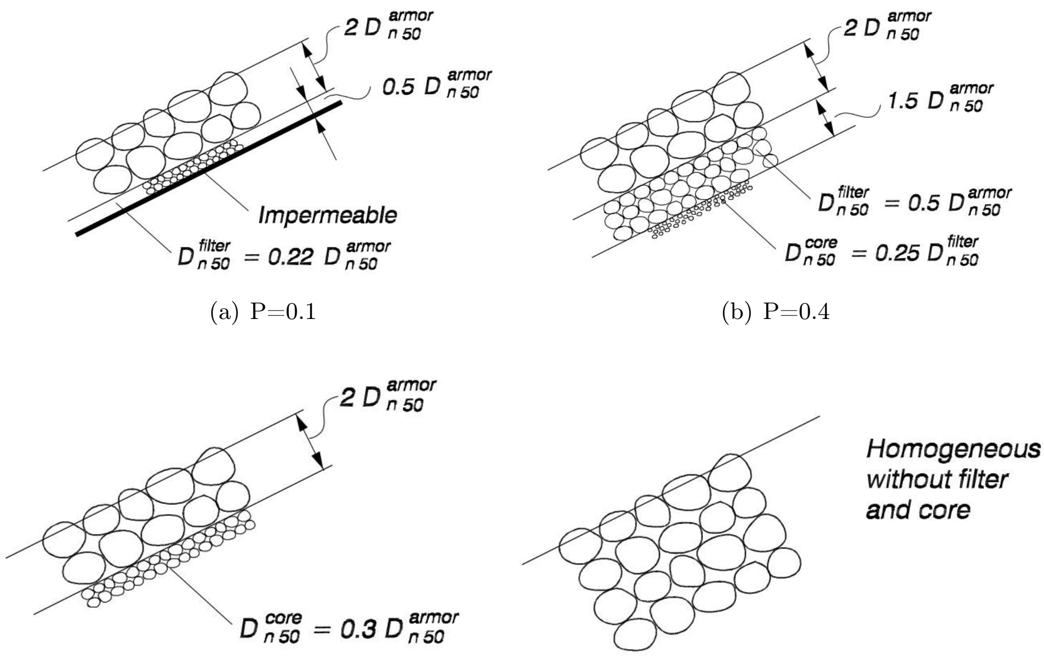
The permeability can be described as the material property that permits movement of water through its pores. It depends on the size of the particles that constitute the filter layer and the core of the structure.

It is an important parameter with respect to the stability or armour layers under wave attack (CIRIA *et al.*, 2007). When a wave impacts on a rubble mound breakwater, the wave penetrates into the rubble mound and energy is dissipated by turbulent flow. If the core is less permeable, increased reflection occurs, leading to higher loads on the armour layer. Therefore a core with low permeability requires larger armour than a core with high permeability. This effect is often not addressed adequately during design and construction (Reedijk *et al.*, 2008). In addition to the design flaws and depending on the production method, large quantities of fines may be present on the core material that will affect the core permeability. The removal of these fines is possible through the use of a strainer however this is an extra effort that contractors tend to avoid. This means that the core permeability of the constructed breakwater may differ substantially from the value assumed by the designer.

In Reedijk *et al.* (2008) the authors warns of the recent trends of using dredged material, sand and sand filled geotextile containers as core material in the construction of breakwaters. These solutions can have a significant economic advantage over the traditional quarry material, specially in the case of areas where the construction of the breakwater can be combined with the dredging of port basins and approach channels. However, the core permeability may be substantially affected (e.g. the application of geotextile in the breakwater core may require larger armour due to reduced core permeability). Moreover, structures with a geotextile filter instead of a granular filter between the armour layer and the core are considered as structures with an impermeable core (CIRIA *et al.*, 2007).

In coastal engineering it is common to consider this parameter as a coefficient without physical meaning, called *Notional permeability factor* (P). Values of P are given by Van der Meer (1988a) ranging between 0.1 for armour on filter on an impermeable core till 0.6 for a structure consisting of armour only (figure 4.7).

The effect of core permeability on interlocking concrete armour unit stability is still unclear and no design guidance is available (Reedijk *et al.*, 2008). However, Burchart *et al.* (1998, cited by Reedijk *et al.*, 2008) conducted some tests on the effect of core permeability on Accropode® armour stability using fine core material ($P \simeq 0.2$) and rough core material ($P \simeq 0.4$). From the results, the authors concluded that the sensitivity to core permeability is higher for Accropodes® than for rock armour.



(a) P=0.1

(b) P=0.4

(c) P=0.5

(d) P=0.6

Figure 4.7: Notional permeability factor (Van der Meer, 1988a)

4.8 Wave incidence angle

For many years, the effect of wave obliquity on the stability of breakwater armour has been hardly investigated, and the estimation of the possible influence of that parameter was often derived from tests not directly related to breakwater stability, such as run-up and wave refraction tests (Galland, 1994).

4.8.1 Approaches

The three reference manuals mentioned in section 3.1 only make small references to the oblique waves attack in the context of armour layer stability. Both the Coastal Engineering Manual, Burchart and Hughes (2011a) and CIRIA *et al.* (2007) refer that the effect of oblique wave approach on armour layer stability has not been sufficiently quantified but some tests seemed to indicate relatively little reduction in damage of rock armoured slopes subjected to oblique wave attack with approach angles up to 60° (relative to normal wave attack), in comparison to heah-on attack. Furthermore, the stability of any rubble-mound structure exposed to oblique wave attack should be confirmed with physical model tests. The BSI (1991) referes that some types of armour units, *e.g.*

Tetrapods and Dolos, are more unstable when attacked by oblique waves and suggest that breakwater roundheads should be tested in three-dimensional wave basins, using waves from various directions. This assertion is supported by Whillock (1977), that made tests on a 1:2 slope armoured with Dolos under regular wave attack with a fixed period. The results of these tests showed a slight decrease in stability with wave attack angles up to 60° (relative to normal wave attack) and then, a large increase in stability for angles greater then 75°. The same trend was also mentioned in Gravesen and Sorensen (1977) for Dolos under random waves for a minimum incidence angle of 45° (relative to normal wave attack), however, the increase in stability for angles higher than 60° was not noticed. The same authors found that, in the case of quarry stone, stability was not much affected for angles ranging from 0° to 45° (relative to normal wave attack), but increased greatly at higher angles (Galland, 1994). Tests preformed by Van de Kreeke (1969) with regular waves with a fixed period and incidence angles ranging from 0° to 90°, for quarry stone with three different slopes (1:1.5, 1:2 and 1:3), revealed the exact same trend. Gamot (1969) presents results from tests conducted in a breakwater armoured with Tretapod blocks. The author stated that the armour stability increased with the increase of the incidence angle for angles higher than 40° (relative to normal wave attack). The author also points to the fact that, once initiated, the damage level progresses faster under oblique waves than under normal waves.

Galland

Specifically to quantify the effect of long-crested oblique waves on rubble mound breakwaters, Galland (1994) carried out a series of tests on four types on armouring units (Quarry stone, Antifer cube, two layers of Tetrapod and one layer of Accropode®), under six angles of wave attack, from 0° to 75° , each 15° (relative to normal wave attack). The following trends could be noticed, for each armour unit:

Antifer cube

- Stability increases with increasing wave obliquity;
- Start of damage is delayed under oblique waves, corresponding to a wave height 50% higher for a 15° , 30° and 45° incidence angle than under head-on waves;
- Damage, once initiated, increases about two times faster for 15° , 30° and 45° incidence angle than under head-on waves;
- For incidence angles higher than 45° , the increase in stability is so high that nearly no damage occurs.

<u>Tetrapod</u>

- Exactly the same trends as noticed in the Antifer cube, but slightly less pronounced and valid mainly for a damage level of less then 10%.

<u>Quary stone</u>

- Start of damage appears to be slightly delayed, but the stone is seen to be not very sensitive to wave obliquity at low damage levels (smaller than 5%);
- For higher damage levels and incidence angles higher than 30° , some trend is noticeable, indicating an increasing stability for increasing incidence angle;
- Stability strongly increases at incidence angles higher than 75° .

<u>Accropode®</u>

- At an incidence angle of 15°, the armour layer behaves similarly as under normal wave attack, with a very sudden failure (characteristic of a one layer unit) which has led to retain a zero-damage criteria for the design of breakwaters armoured with this unit;
- At higher angles, after some damage, units rearrange so that the armour is stable again and no more damage occur.

From the result of these tests, it was possible to introduce the concept of *equivalent significant wave height* ($H_{S,\beta}$), defined in equation 4.3, instead of the normal significant wave height (H_S), as a way of taking into account the influence of oblique waves in existing formulas. The factor X is different for each armour unit (0.6 for Antifer, 0.3 for Tetrapod, 0.25 for Rock and 1.0 for Accropode®).

$$H_{S,\beta} = H_S \cos_\beta^X \tag{4.3}$$

For a specific wave height, the weight of the armour unit can be reduced for oblique waves. The reduction factor (F) for the required armour weight compared to conditions with perpendicular wave attack can be expressed by equation 4.4 (where $W_\perp$ is the armour unit weight for normal waves by equation 3.1 and W_β is the armour unit weight for oblique waves by equation 3.1). The resulting reduction factor of applying the *equivalent normal wave height* ($H_{S,\beta}$) can be seen in figure 4.8, for each of the studied units.

$$F = \frac{W_\beta}{W_\perp} \tag{4.4}$$

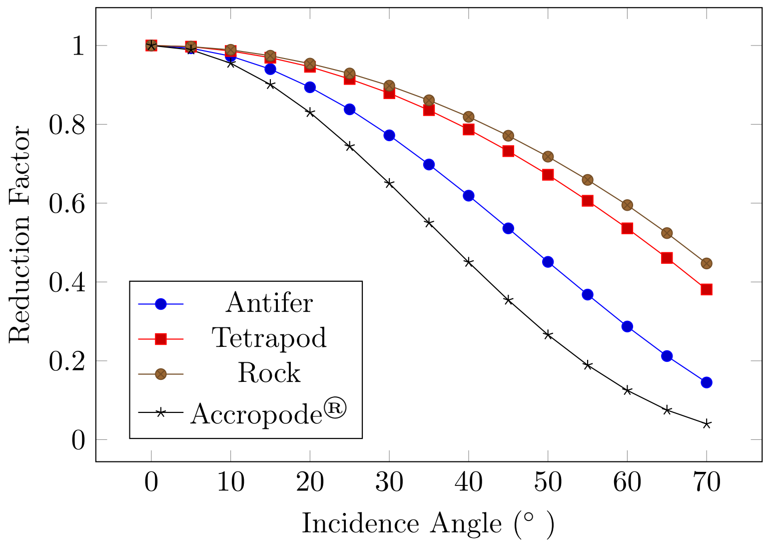


Figure 4.8: Reduction factor for wave obliquity, based on Galland (1994)

Gent

Using the same *equivalent significant wave height* ($H_{S,\beta}$) approach, Van Gent (2003b) and Wolters and Van Gent (2011) preformed physical tests in structures with rock and cubes armour layers, under oblique wave attack and with permeable and impermeable cores. These tests resulted in new values for the X coefficient in equation 4.3. These values are presented in table 4.8. The resulting reduction factor, expressed by equation 4.4, of applying the *equivalent significant wave height* ($H_{S,\beta}$) can be seen in figure 4.9, for each of the tested conditions. The results from this study showed that the previous formulation of Galland (1994), to include the wave obliquity, underestimate the effects of oblique wave attack.

Table 4.8: Wave obliquity coefficient X for the equivalent wave height ($H_{S,\beta}$)

Core type	Rock	Cubes (single layer)	Cubes (double layer)
Permeable	1.05^a	2.5^b	1.5^a
Impermeable	1.05^a	-	0.95^a

[a] For wave attack angles 0° - 70°
[b] Only for wave attack angles 0° - 45°

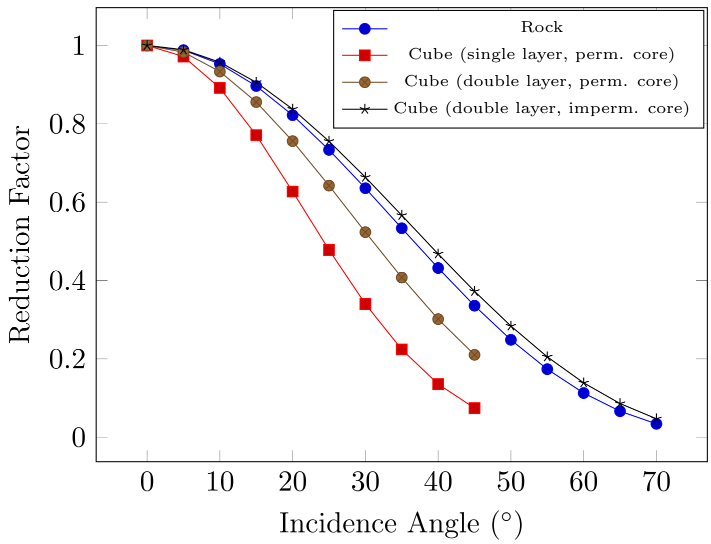


Figure 4.9: Reduction factor for wave obliquity, based in Wolters and Van Gent (2011)

Yu *et al.*

In a different approach to the wave obliquity problematic, and intended to be used specifically with Hudson's equation (equation 3.1), Yu *et al.* (2002) introduced the concept that the effect on armour unit stability can be described with the variation of the stability coefficient (K_D) in Hudson's equation. The proposal is the result of an extensive 3D model test program on four types of armour units (Dolosse, Accropode®, Hollow square and quarry stones), under five angles of wave attack (0° , 15° , 30° , 45° and 60°), for regular and irregular waves, for long and short-crested waves. The results of these tests revealed some of the same trends mentioned by previous authors and some new findings, like:

- The stability increases with increasing wave obliquity;
- Under the action of oblique waves, start of damage is delayed;
- The damage increases faster under oblique wave action than that under normal wave action;
- For quarry stones, there is a similar reduction in required rock size for long-crested waves and for short-crested waves;
- For short-crested waves, the stability of concrete armour units is better than for long-crested waves, leading to a required armour size for oblique waves that is smaller for long-crested waves than for short-crested waves.

Based on this study, the authors propose an *equivalent stability coefficient* ($K_{D,\beta}$), defined in equation 4.5, as a way of taking into account the influence of oblique waves in Hudson's equation. The factor X is different for each armour unit (1.02 for Dolosse, 1.47 for Hollow square, 1.55 for Quarry stone and 2.3 to Accropode®).

$$K_{D,\beta} = K_D \cos_{\beta}^{-X} \tag{4.5}$$

The resulting *equivalent stability coefficient* is greater than the stability coefficient on which is based, and the difference increases with increasing wave obliquity. This can be better demonstrated by comparing the Stability Number (N_S), defined in equation 4.6, for both oblique and normal incident waves.

$$N_S = (K_D \cot \alpha)^{\frac{1}{3}} \tag{4.6}$$

Where:

- K_D - Hudson's stability parameter
- α - slope angle (°)

Similar to the case of the last approach, a reduction factor can be applied to the size of the armour unit for oblique waves. This reduction factor (F) for the required armour size compared to conditions with perpendicular wave attack can be expressed by equation 4.7. The resulting reduction factor of applying the *equivalent stability coefficient* ($K_{D,\beta}$) can be seen in figure 4.10, for each of the studied units in a wide range of wave incidence angles.

$$F = \frac{N_{S,\perp}}{N_{S,\beta}} = \frac{\left(\frac{H_S}{\Delta D}\right)_{\perp}}{\left(\frac{H_S}{\Delta D}\right)_{\beta}} = \frac{D_\beta}{D_\perp} \tag{4.7}$$

Where:

- $N_{S\perp}$ - Stability number for normal waves
- $N_{S\beta}$ - Stability number for oblique waves

It is important to mention that the reduction factor calculated for this approach is not directly comparable to the factors considered for the previous authors, because in the approaches from Galland (1994) and Van Gent (2003a) the reduction factor is relative to the weight of the unit, and in the approach from Yu *et al.* (2002), the reduction factor is relative to the unit nominal diameter.

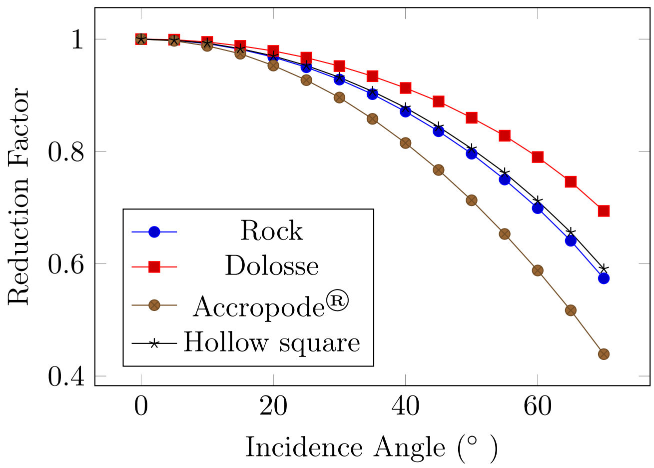


Figure 4.10: Reduction factor for wave obliquity, based in Yu *et al.* (2002)

Gent (2014)

More recently, Van Gent (2014) re-analyzed the data presented in Galland (1994), Yu *et al.* (2002) and Wolters and Van Gent (2011). Considering that the amount of energy (E_β) that reaches a specific stretch of the structure can be described by equation 4.8, the author states that for a specific wave height, the armour unit size can be reduced for oblique wave attack using the reduction factor (F) expressed in equation 4.9, in which the X factor depends on the type of armour unit and on the author. These values are presented in table 4.9.

$$E_\beta = E_\perp \cdot \cos \beta \tag{4.8}$$

$$F = \frac{N_{S,\perp}}{N_{S,\beta}} = \frac{\left(\frac{H_S}{\Delta D}\right)_\perp}{\left(\frac{H_S}{\Delta D}\right)_\beta} = \frac{D_\beta}{D_\perp} = \cos^X \beta \tag{4.9}$$

Where:

- $N_{S\perp}$ - Stability number for normal waves
- $N_{S\beta}$ - Stability number for oblique waves
- H_S - Wave height at the toe of the structure (m)
- Δ - Relative density of the unit material
- $D_\perp$ - Armour unit diameter for normal waves (m)
- D_β - Armour unit diameter for oblique waves (m)

Table 4.9: Wave obliquity coefficient X for armour stability from various authors

Author	Rock	Cubes	Antifer	Tetrapod	Dolosse	Accropode®
Galland (1994)	0.25	-	0.6	0.3	-	1
Yu *et al.* (2002)	1.157	-	-	-	1.007	1.32
Wolters and Van Gent (2011)	1.1	0.95	-	-	-	-

However, equation 4.9 leads to a no damage situation, independent of the conditions, for a wave angle of $\beta = 90°$. Although the damage is much smaller than for perpendicular waves, tests suggest that such parallel waves may still lead to damage on the armour layer. Therefore, equation 4.9 cannot be used in the full range of wave obliquity and the author proposes a new formula, expressed in equation 4.10, that follows the observed interactions of oblique waves and the armour layer, such as, a small influence for small and very large attack angles ($> 70°$) and larger influence for larger attack angles.

$$F = (1 - c_\beta) \cos^2_\beta + c_\beta \tag{4.10}$$

The coefficient c_β was obtained from test results and is dependent of the type of wave (long-crested waves or short-crested waves) and on the unit type and placement. The optimal values of c_β for rock are $c_\beta = 0.35$ for long-crested waves and $c_\beta = 0.42$ for short-crested waves. As for the cubes, the optimal values were found to be $c_\beta = 0.35$ for cubes in a double layer and $c_\beta = 0$ for cubes in a single layer (in this case, only tests up to $\beta = 45°$ were performed). The shape of equation 4.10 for each value of c_β is shown in figure 4.11.

As demonstrated by figure 4.11, equation 4.10 leads to a smaller obliquity influence for small wave angles and for the largest wave angles, and a maximum influence at $\beta = 45°$. The reduction factor given in equation 4.10 can be used in combination with existing formulas to predict the required size of the armour unit under oblique wave attack. However, the author advises that since the calibration of the formula is based on damage levels above a specific limit, it cannot be used accurately to predict the transition from "no damage" to "initial damage". Therefore, the equation is more effective when applied to structures in which the design is based on higher damage levels, such as $S_d \geq 2$ or $N_{OD} \geq 2$. Comparing the tests with an impermeable core with the tests with a permeable core for 1:1.5 slopes indicates that the influence of oblique waves can be stronger for structures with an impermeable core. However, for a 1:2 slope with an impermeable core the influence seems to be somewhat weaker than predicted with the method. Therefore, the author concludes that the influence of the permeability of

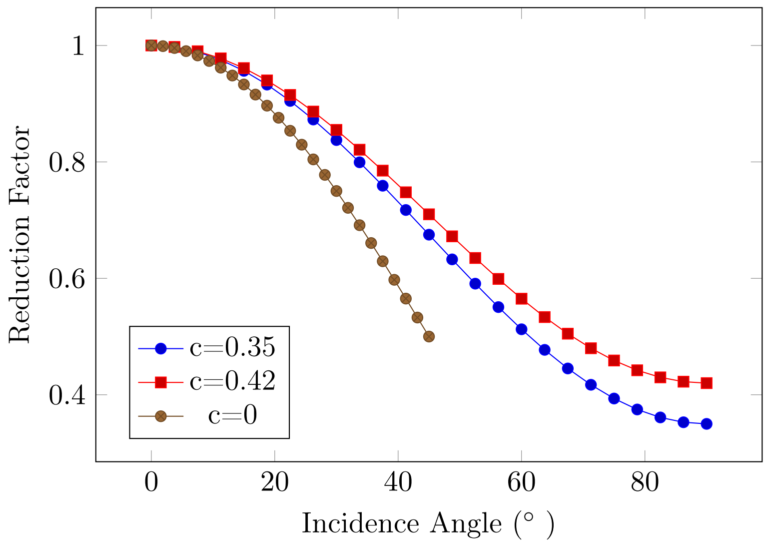


Figure 4.11: Expression for the reduction in required diameter for oblique waves (Van Gent, 2014)

the core on the reduction factor is not significant (Van Gent, 2014). The results show that for cubes there is no important difference between attack with long-crested waves and short-crested waves, contrary to the findings of Yu *et al.* (2002), that for the tested concrete armour units (not specifically cubes), the influence of wave obliquity for short-crested waves is weaker than for long-crested waves. For rocks, Yu *et al.* (2002) found no important difference between long-crested waves and short-crested waves, however, data from Van Gent (2014) show that for rock slopes the influence of wave obliquity is stronger for long-crested waves than for short-crested waves.

4.8.2 Discussion

Considering the multiple approaches and the dispersed data throughout the bibliography, it is hard to put in perspective the comparison between different authors. Figure 4.12 represents a plot of the reduction factor for rock armour, for each of the considered authors. The reduction factor (F) is based on equation 4.9 with the coefficients expressed in table 4.9 for rock, with the exception of the reduction factor for Van Gent (2014), that is based on equation 4.10 with the coefficient for long-crested waves ($c_\beta = 0.35$).

Figure 4.12 shows that for rock slopes, the results of Yu *et al.* (2002), Wolters and Van Gent (2011) and Van Gent (2014) match rather well until $\beta = 60°$, even considering that the tests from Yu *et al.* (2002) were performed with a uniform rock material instead of a standard rock gradient. The limit angle to use the formula by Wolters and Van Gent (2011) is $\beta = 70°$ and at this angle this formula gives a reduction about 12% larger than

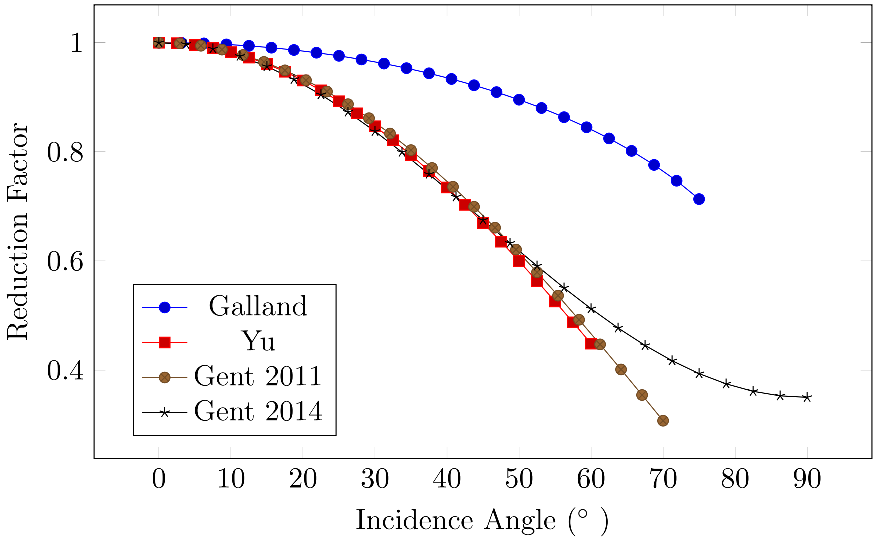


Figure 4.12: Rock armour units reduction factor for various authors

the formula by Van Gent (2014). For larger wave angles, Wolters and Van Gent (2011) would lead to an overestimate of the reduction effect, while applying the formula from Van Gent (2014) would lead to a more conservative estimate. The formula by Galland (1994) results in a much smaller effect of wave obliquity on rock armour layers than the results from the other studies (Van Gent, 2014).

Figure 4.13 represents a plot of the reduction factor for the cube armour unit for equation 4.9 by Wolters and Van Gent (2011) and equation 4.10 by Van Gent (2014). The figure shows that between $\beta = 0°$ and $\beta = 60°$, there is no significant difference between the equations. At the limit angle of $\beta = 70°$, the difference keeps on being only in the range of 6%. However, by extrapolating equation 4.9 for angles greater than $\beta = 70°$, equation 4.10 proves to be much more conservative. Therefore, equation 4.10 is recommended over equation 4.9, as it allows a broader range of attack angles and has been tested in an armour layer constituted by only one layer of cubes. It is believed that the relative large influence of obliquity for cubes in a single layer is related to the rather smooth surface of the armour layer (Van Gent, 2014).

Figure 4.14 represents a plot of the reduction factor for concrete armour for each of the mentioned authors. As in the case of the rock unit, the reduction factor (F) for all the authors is based on equation 4.9 with the coefficients expressed in table 4.9 for each type of unit, with the exception of the reduction factor for cubes by Van Gent (2014), that is based on equation 4.10 with the coefficient for cubes placed on two layers ($c_\beta =$ 0.35).

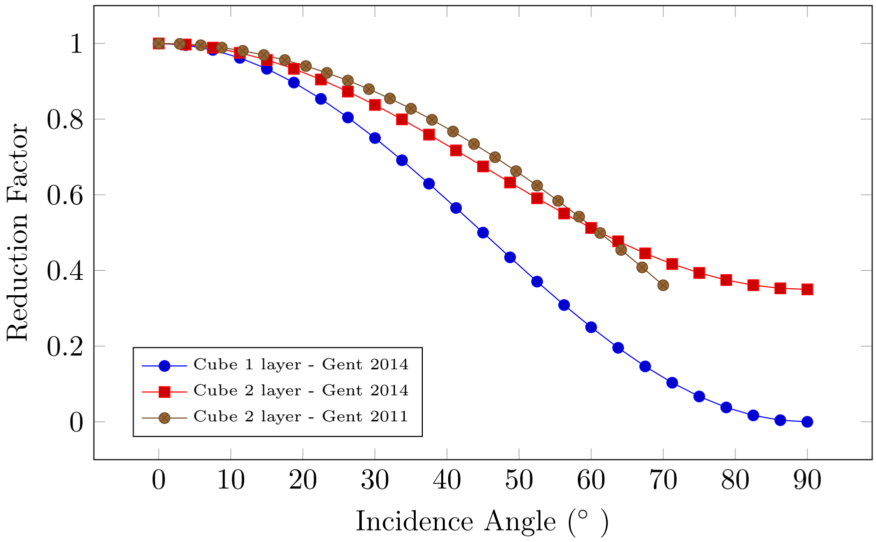


Figure 4.13: Cube units reduction factors by different approaches

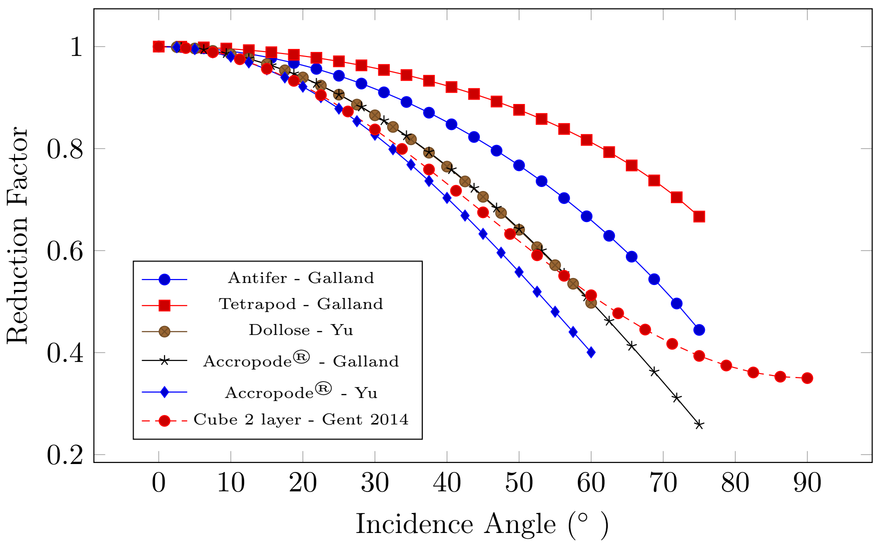


Figure 4.14: Concrete units reduction factors by different authors

Figure 4.14 shows that the influence of wave obliquity is less for the Antifer and Tetrapod units than for the remaining concrete units. The figure also indicates that for angles outside the range of validity of equation 4.9 ($\beta > 70°$), the results are likely to lead to an overestimate of the influence of wave obliquity and to smaller armour material then necessary (Van Gent, 2014).

In the design process of the Angeiras breakwater, described in section 6.1.2 of the this paper, the wave obliquity influence is described as a coefficient that affects the slope angle of the structure. The concept behind this approach by CONSULMAR (2011a) is similar to the one presented by Van Gent (2014) with equation 4.8. The same principle is applied by distributing the energy of the incident wave into a longer horizontal stretch of the structure's slope compared to the stretch considered for a wave, with the same characteristics, attacking the structure perpendicularly. Considering a generic slope ($V : H$), the *equivalent slope* ($V : H_\beta$) is gentler, due to an increase in the horizontal component, expressed by equation 4.11.

$$H_\beta = \frac{H}{\cos\beta} \tag{4.11}$$

This is the most simplistic approach to the wave obliquity problematic. It only accounts for the wave attack angle and it does not consider variables such as unit type, placement, number of layers, type of wave or core permeability. Therefore, this approach is not suitable for units in which the efficiency is highly dependent on the interlock capability (*e.g.* Dolosse and Tetrapod). Equation 4.11 is aimed particularly at units in which the resistance is achieved by its own weight. Similar to the approach presented by Galland (1994), for a specific wave height, the weight of the armour unit can be reduced for oblique waves. The reduction factor (F) for the required armour weight compared to conditions with perpendicular wave attack can be expressed by equation 4.4.

Whereas both the approaches from Van Gent (2014) and CONSULMAR (2011a) define the reduction factor depending on the Stability number (N_s), these approaches are directly comparable. This comparison is displayed in figure 4.15 for rock armour units and in figure 4.16 for selected concrete armour units.

Figure 4.15 shows that, for rock armour layer, the approach from CONSULMAR (2011a) is much more conservative than all others, with the exception of the approach from Galland (1994). Between these two approaches, there is no significant difference in the results, being the largest difference in the range of 5%. Other approaches provide a 25% larger reduction at $\beta = 45°$, giving a significant reduction in the necessary unit size. Even considering that these approaches are a result of more recent studies, such large reductions should be carefully considered.

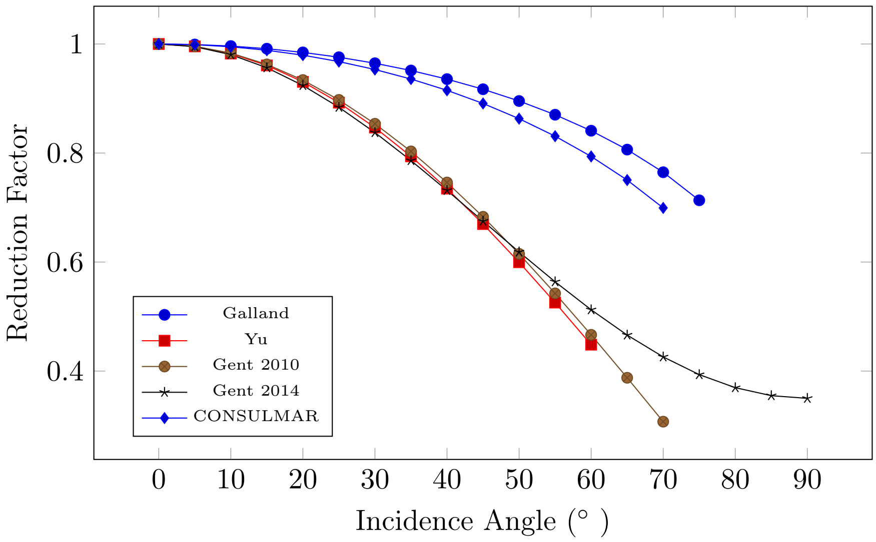


Figure 4.15: Comparison of the reduction factor by CONSULMAR with other authors (rock)

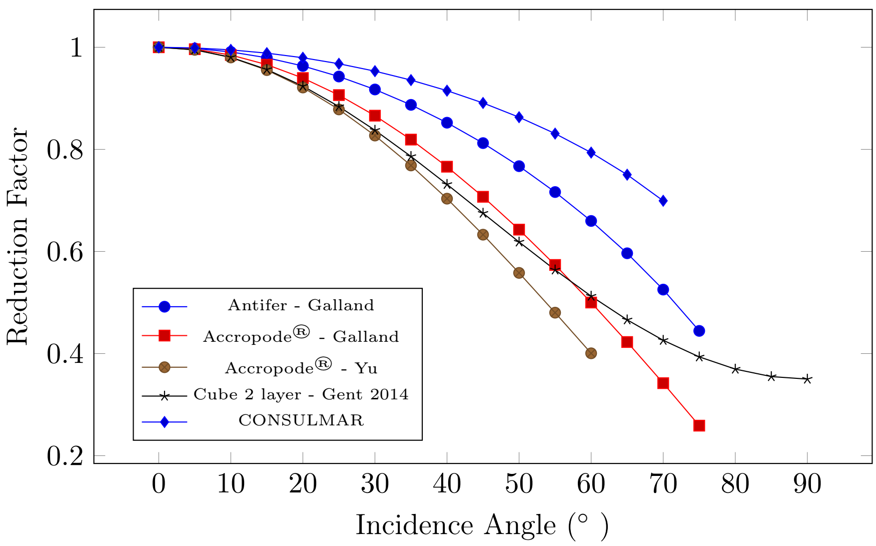


Figure 4.16: Comparison of the reduction factor by CONSULMAR with other authors (concrete)

Figure 4.16 shows that, for concrete armour layer units, the approach from CONSULMAR (2011a) continues to be the most conservative but by a smaller margin than on the rock armour units. The less conservative approach is Yu *et al.* (2002) and provides a reduction in size about 13% larger at $\beta = 45°$ and 40% larger at the limit angle of $\beta = 60°$). However, caution is recommended when dealing with concrete armour units, due to the greater uncertainty on defining its stability (*e.g.* the large range of K_D values recommended for the Antifer unit considering the influence of the placement method described in section 4.1 of this document).

Considering the large number of approaches presented, a summary is presented in table 4.10 in order to facilitate the inclusion of the wave attack angle parameter in the design of coastal structures.

4.9 Other

Besides the ones emphasized, other parameters affect the stability of the armour layer:

- Number of layers comprising the armour - the lower the number of layers, the higher is the weight required for the individual units. This is explained by redundancy, in the event of a unit of the top layer being removed, there is still more armour units bellow, protecting the structure. The same doesn't apply when there are fewer layers. If a one-unit armour layer is considered, it will require heavier units to ensure stability (USACE, 1984);

- Part of the structure (trunk or head) - under all wave conditions, regardless of wave direction, a segment of the structure's head is always exposed to direct wave attack. For this reason, the head normally sustains more extensive and frequent damage than the trunk and will require heavier units to ensure stability (USACE, 1984);

- Wave Period - the parameter K_D of the Hudson equation is found strongly dependent on the wave period (Yoo, 2010). This conclusion was taken after a series of physical model tests using the Sealock armour unit and, it should be tested for other units to determine the relevance of this parameter for the final value. The values of Hudson parameter are found to become smaller for longer wave period, that is, longer waves are found more damageable to the stability of armoured breakwaters.

Table 4.10: Wave angle summary table

Unit	Author	Formula	Conditions
Rock	Galland (1994)	$H_{S,\beta} = H_S \cos_\beta^{0.25}$	$0° > \beta \geq 70°$
	Wolters and Van Gent (2011)	$H_{S,\beta} = H_S \cos_\beta^{1.05}$	$0° > \beta \geq 70°$
	Yu *et al.* (2002)	$K_{D,\beta} = K_D \cos_\beta^{-1.55}$	$0° > \beta \geq 60°$ Hudson eq.
	Van Gent (2014)	${}^aF = 0.65 \cos_\beta^2 +0.35$	Long-crested[b]
		${}^aF = 0.58 \cos_\beta^2 +0.42$	Short-crested[c]
Antifer	Galland (1994)	$H_{S,\beta} = H_S \cos_\beta^{0.6}$	$0° > \beta \geq 75°$
Tetrapod	Galland (1994)	$H_{S,\beta} = H_S \cos_\beta^{0.3}$	$0° > \beta \geq 75°$
Accropode®	Galland (1994)	$H_{S,\beta} = H_S \cos_\beta$	$0° > \beta \geq 75°$
	Yu *et al.* (2002)	$K_{D,\beta} = K_D \cos_\beta^{-2.3}$	$0° > \beta \geq 60°$ Hudson eq.
Dolosse	Yu *et al.* (2002)	$K_{D,\beta} = K_D \cos_\beta^{-1.02}$	$0° > \beta \geq 60°$ Hudson eq.
Hollow square	Yu *et al.* (2002)	$K_{D,\beta} = K_D \cos_\beta^{-1.47}$	$0° > \beta \geq 60°$ Hudson eq.
Cube (single layer)	Van Gent (2003b)	$H_{S,\beta} = H_S \cos_\beta^{2.5}$	$0° > \beta \geq 45°$ Permeable
	Van Gent (2014)	${}^aF = \cos_\beta^2$	Permeable
Cube (double layer)	Wolters and Van Gent (2011)	$H_{S,\beta} = H_S \cos_\beta^{1.5}$	$0° > \beta \geq 45°$ Permeable
	Wolters and Van Gent (2011)	$H_{S,\beta} = H_S \cos_\beta^{0.95}$	$0° > \beta \geq 70°$ Impermeable
	Van Gent (2014)	${}^aF = 0.65 \cos_\beta^2 +0.35$	-

[a] F is defined by equation 4.7;
[b] Long-crested waves;
[c] Short-crested waves.

Chapter 5

Sensitivity analyzes

5.1 General considerations

As mentioned before, the Hudson's equation is widely used worldwide due to its simplicity and proven results from decades of appliance to the design of breakwater armor layers. However, this simplicity comes at the expense of impose that the designer chooses a *stability coefficient* (K_D) that, alone, should reflect all the governing parameters affecting the stability of the structure, taking those included in the equation itself. The second option presented in this document is the Van der Meer formula, that takes more parameters into account explicitly in the equation.

In this chapter, sensitivity analysis are performed in order to evaluate the influence of the stability parameters included in the Van der Meer formula on the *stability coefficient* (K_D), and on the overall stability of the structure.

As presented in chapter 3, the Van der Meer formula varies according to the conditions and armour units adopted for the design. In the present analysis, only the formulas for deep water and rock armour layers are considered (equations 3.2 and 3.3).

In order for the Van der Meer formulas to be directly comparable to Hudson formula, Van der Meer (1988a) proposes a modification of the Hudson formula so that the damage level could be included trough the introduction of the S_d parameter (see section 4.5) in the formula. The proposed formula is presented as follows:

$$\frac{H_S}{\Delta D_{n50}} = 0.7(K_D \cot\alpha)^{1/3} S_d^{0.15} \tag{5.1}$$

The relation between the modified Hudson formula and the Van der Meer formulas is made by matching the stability number, defined in equation 2.1. This relation is expressed in equations 5.2 and 5.3, for plunging and surging waves, respectively. The distinction between plunging and surging waves is made using the method presented in

Van der Meer (1988a) and discussed in section 3.4 of this document.

$$0.7(K_D \cot\alpha)^{1/3} S_d^{0.15} = 6.2P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{5.2}$$

$$0.7(K_D \cot\alpha)^{1/3} S_d^{0.15} = 1.0P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\; \xi_m^{P} \tag{5.3}$$

The analysis of the influence of each individual parameter on the *stability coefficient* (K_D) is performed by plotting the parameter with the resulting *stability coefficient* value and, in order to accomplish this, it is necessary to solve the equations 5.2 and 5.3 for K_D:

$$K_D = \frac{\left(\frac{8.86P^{0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5}}{S_d^{0.15}}\right)^3}{\cot\alpha} \tag{5.4}$$

$$K_D = \frac{\left(\frac{1.43P^{-0.13}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\;\xi_m^{P}}{S_d^{0.15}}\right)^3}{\cot\alpha} \tag{5.5}$$

The influence of each parameter in the overall stability of the structure is performed by plotting the parameter with the *stability number* (N_S), calculated trough equation 5.1, considering the previously calculated *stability coefficient* in equations 5.4 and 5.5.

$$N_S = 0.7(K_D \cot\alpha)^{1/3} S_d^{0.15} \tag{5.6}$$

The use of the Van der Meer formulas implies that the boundary conditions for the Van der Meer formula parameters (expressed in table 3.2) must be respected.

Accordingly to Van der Meer (1988a), the governing variables that define the static stability of a structure are:

- Wave height (H), expressed by the stability number (N_S);
- Wave period (T), expressed by the wave steepness parameter (s_m);
- Storm duration, expressed by the number of waves (N);
- Damage level (S_d);
- Permeability, expressed by the notional permeability parameter (P);
- Slope inclination;

- Shape of the stone;
- Water depth in front of the structure;
- Angle of wave attack.

The wave height is not analyzed as an individual parameter due to its influence on all the other parameters trough the *mean surf parameter* (ξ_m). The considered wave height is the *specific wave height*, this being the recommended parameter for the Hudson formula by the USACE (1977) and for the Van der Meer formulas for deep water conditions. The wave period is not analyzed as an individual parameter but as a linear dependency of the wave height. This proportionality is based on the findings presented in Coelho (2005) as a result of the relation between the wave height and period of the waves recorded in Leixões buoy, between 1981 and 2003. This relation is presented in equation 5.7.

$$T = 1.21H + 6.92 \tag{5.7}$$

The range of values considered for the storm duration parameter is between $N = 1000$ and $N = 7500$ waves. Accordingly to CIRIA *et al.* (2007), the maximum number of waves to be inserted in equations 5.2 and 5.3 is 7500. After this number of waves the armour layer is considered to have reached an equilibrium. Conditions with a larger number of waves may be considered, but the maximum number to be used is 7500. For a number of waves smaller than 1000, its influence on the layer stability must be calculated with a different approach than the one considered in Van der Meer (1988a), this is because the development of the damage, appears for small numbers of waves ($N < 1000$) to be linear with N instead of proportional to the square root of N (CIRIA *et al.*, 2007), therefore it is not going to be included in this analysis.

In relation to the damage level, the considered range of values is from $S_d = 2$ (initial damage or no-damage in the Hudson formula) to $S_d = 15$ (failure), passing by the values $S_d = 6$ to $S_d = 9$ (intermediate damage). These damage level values are presented in Van der Meer (1988a) and are dependent on the layer slope inclination.

The notional permeability parameter has a fixed range of $P = 0.1$ to $P = 0.6$. Considering that $P = 0.1$ is the characteristic value of a structure with a completely impermeable core, this value will not be used in this analysis. Structures with $P = 0.1$ require special attention in the design. Completely homogeneous structures ($P = 0.6$) are not common in traditional breakwater design. Therefore, the range considered in this analysis for the notional permeability parameter is from $P = 0.2$ to $P = 0.5$.

The slope inclination parameter was given a variation between 1:3 (V:H) and 3:4

(V:H) by characterizing the more usual slopes for breakwaters.

The stone shape parameter is not subject to analysis because it is already covered in the recommended K_D values and it is not an explicit parameter in the Van der Meer formula. The water depth at the toe of the structure is not considered because this analysis is only valid for structures with deep water implantation. The angle of wave attack is already discussed in section 4.8 and will not be addressed further.

When analyzing a single parameter, the remaining parameters have the following default values:

- N=4000;
- P=0.4;
- S_d=2;
- Slope inclination = 1:2 (V:H).

In order to compare the usual stability coefficient values with the ones found in this analysis for each of the studied parameters, two values were retrieved from the recommended K_D values by USACE (1977). These are the recommended values for a common breakwater armour layer: trunk section, two layers, smooth rounded rocks ($K_D = 2.4$) and rough angular rocks ($K_D = 4$).

5.2 Discussion

5.2.1 Permeability

In the Hudson formula, the permeability is an important parameter that is not directly implicit in the formula. It is one of the many parameters included in the stability coefficient. The intent of this analysis is to have an idea of the variation on the stability coefficient, induced by the permeability. Therefore, two analysis are performed: figure 5.1 shows a plot of K_D as a function of the permeability (P) for three different values of wave height (H_S) and, figure 5.2 shows a plot of K_D as a function of the wave height (H_S) for three different permeability values.

The K_D value is found to be considerably dependent on the permeability factor. In the worst case, for $H_S = 2$ the variation in the permeability from $P = 0.2$ to $P = 0.5$, causes an increase in K_D of about 150% (figure 5.1).

Figure 5.1 clearly demonstrates a stronger influence of the permeability on the K_D value for surging waves than for plunging waves, as the curve for $H_S = 2$ (that never reaches the plunging conditions) and the remaining curves for $P > 0.45$ are steeper than

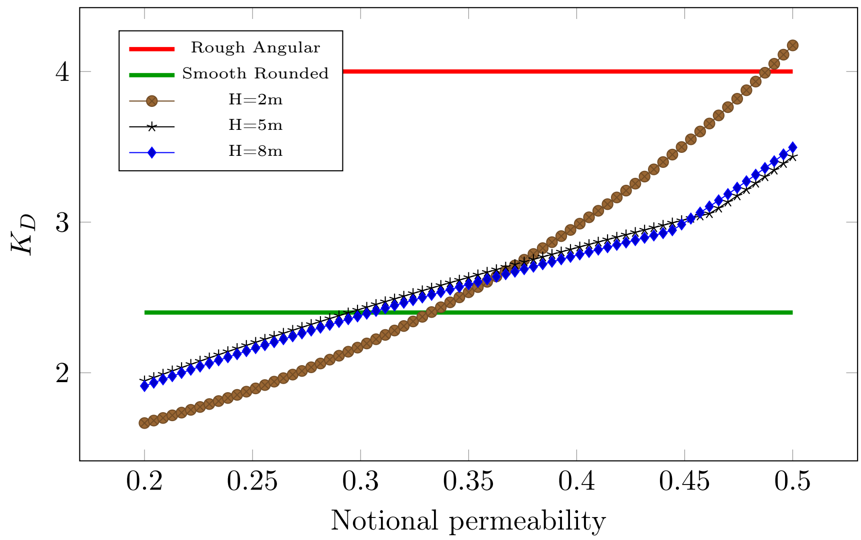


Figure 5.1: K_D *vs* Nominal permeability (P), for different wave heights

the $H_S = 5$ and $H_S = 8$ for $P < 0.45$. $P = 0.45$ is the transition point from plunging to surging waves. However, as seen as figure 5.2 demonstrates, the minimum K_D value, for which the larger blocks are necessary, is found for collapsing waves (transition from surging and plunging waves). The explanation for this is that in the plunging region the wave run-up after breaking is decisive for stability and in the surging region the decisive load is the wave run-down. In the collapsing region, both run-down and run-up forces are high, which results in the most demanding situation.

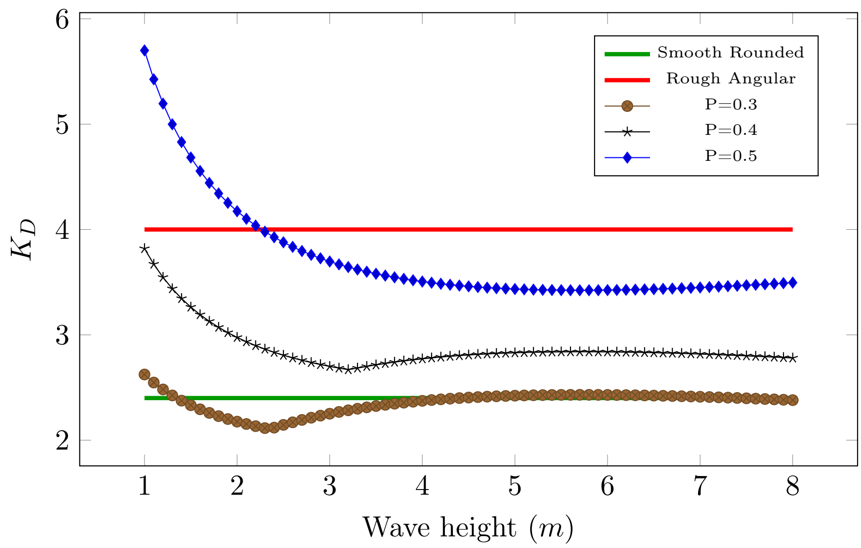


Figure 5.2: K_D *vs* Wave height, for different permeability values

By analyzing figure 5.2, the permeability seems to affect the wave height at which the wave passes from plunging to surging by means of the *critical surf parameter* (ξ_{cr}), as the point moves to the right as the permeability increases. However figure 5.1 shows that the main factor influencing the transition is the wave height or the wave period by means of the *mean surf parameter* (ξ_m), as the transition permeability is the same for $H_S = 5$ and $H_S = 8$.

Using the data plotted in figure 5.2, the K_D variation between different permeability values for each of the wave heights was calculated (equation 5.8). The average of the resulting values are then calculated and presented in figure 5.3.

$$\Delta_{K_D} = \left(\frac{K_{D_{Pi}} - K_{D,Pj}}{K_{D,Pi}}\right)_{H_k} \tag{5.8}$$

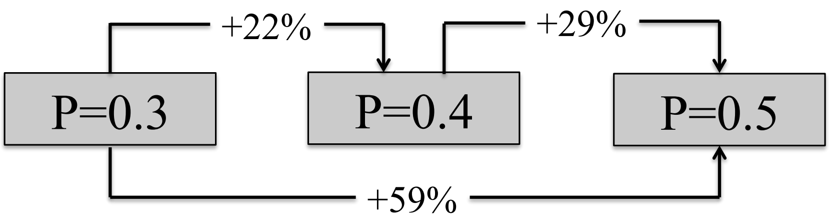


Figure 5.3: K_D variation for different values of permeability

For structures with low permeability ($P < 0.4$), it is possible that the recommended value for rough angular stones underestimates the layer stability, which may cause the structure to be tested with smaller units than necessary for the design conditions. This would require a new design and a new series of physical model tests. On the other hand, the recommended stability coefficient value for smooth rounded rock units could be conservative in relation to structures with relatively permeable cores ($P > 0.4$), which would lead to larger stones than necessary to be placed on the armour layer, increasing the cost of the structure.

5.2.2 Storm duration

By analyzing figure 5.4 and considering that for $H_S = 2$ the waves are of the surging breaking type and the waves are of the plunging breaking type for $H_S = 5$ and $H_S = 8$, it is clear that the influence of the number of waves has very little dependency or is independent of the type of wave attacking the structure. Figure 5.5 shows that throughout all the range of wave heights, the absolute difference in the stability coefficient value, between two considered number of waves (*e.g.* $N = 1000$ and $N = 4000$) is always the same. The relative difference between in the stability coefficient value between the

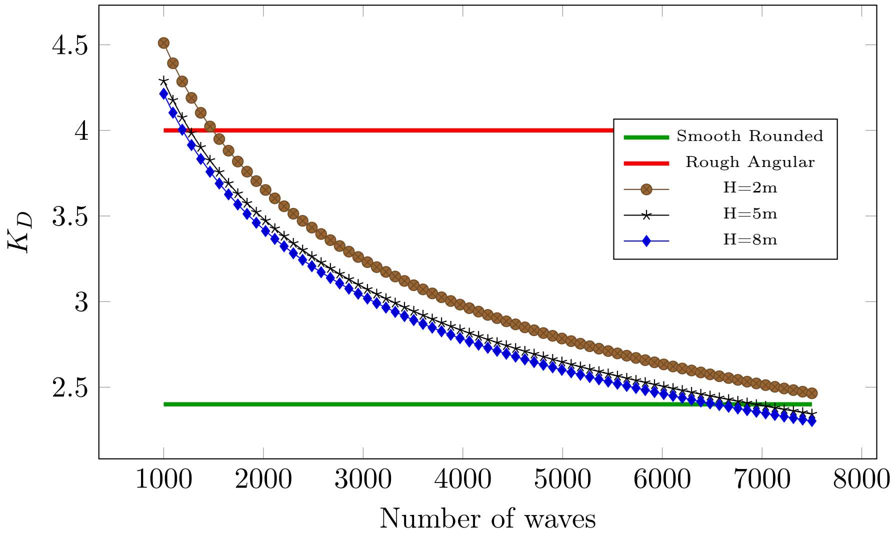


Figure 5.4: K_D *vs* Number of waves (N), for different wave heights

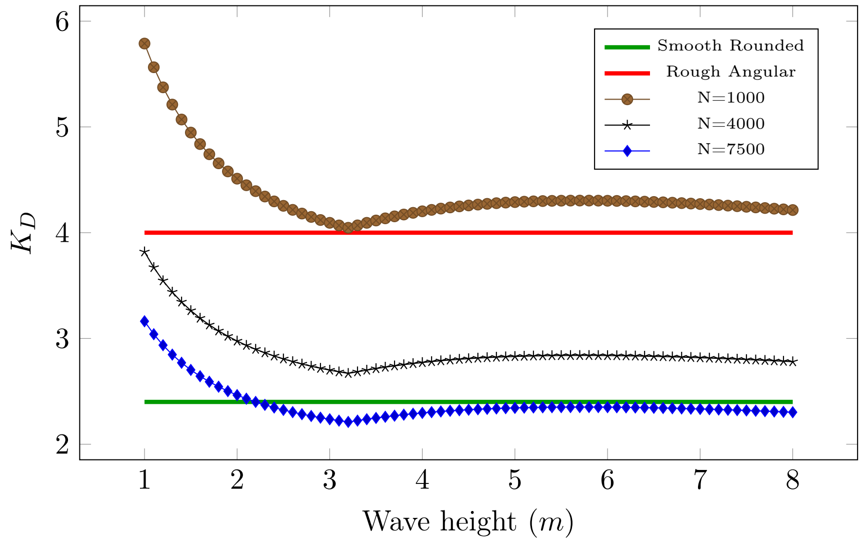


Figure 5.5: K_D *vs* Wave height, for different number of waves

considered number of waves are calculated using equation 5.9 and presented in figure 5.6.

$$\Delta_{K_D} = \frac{K_{D_{Ni}} - K_{D_{Nj}}}{K_{D_{Nj}}} \tag{5.9}$$

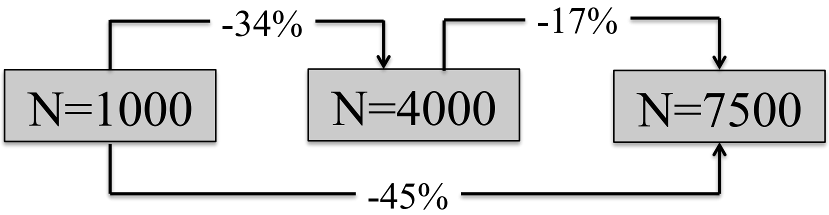


Figure 5.6: K_D variation for different number of waves

Figure 5.4, supported by figure 5.6, shows a clear decrease in the influence of the storm duration over the stability coefficient with the increase of the number of waves. This interpretation is consistent with the statement in CIRIA *et al.* (2007), that longer storm durations can be considered, but the maximum number of waves to use is 7500.

As seen in figure 5.5 and similarly to the findings regarding the permeability parameter, the recommended stability coefficient value, for rough angular stones, could underestimate the stability of the layer, when compared to the correspondent conditions calculated by the Van der Meer formulas for a long duration storm ($N > 4000$). On the other hand, considering short duration storms ($N < 3000$), the recommended value for smooth round rocks could be conservative.

5.2.3 Damage level

The damage level is one of the important parameters that is not implicit in the original Hudson formula. However, this parameter was included in the modified Hudson formula in Van der Meer (1988a). Therefore, this parameter can be treated as independent from the stability coefficient and its influence on stability of the structure should be analyzed through the stability number (N_S).

The interest of analyzing both approaches (modified Hudson and Van der Meer formulas) is that each treats the influence of the damage parameter differently. Figure 5.7 shows the Van der Meer formula appears to give more influence to the damage level parameter than the modified Hudson formula, as the curves representing the Van der Meer formulas are steeper. In figure 5.7 is also visible that as the stability number is slightly higher for $H = 2m$ than for $H = 5m$ and $H = 8m$, that are coincident. This difference could be caused by the fact that $H = 2m$ is in the surging break type region, and the other wave heights considered are in the plunging break type region.

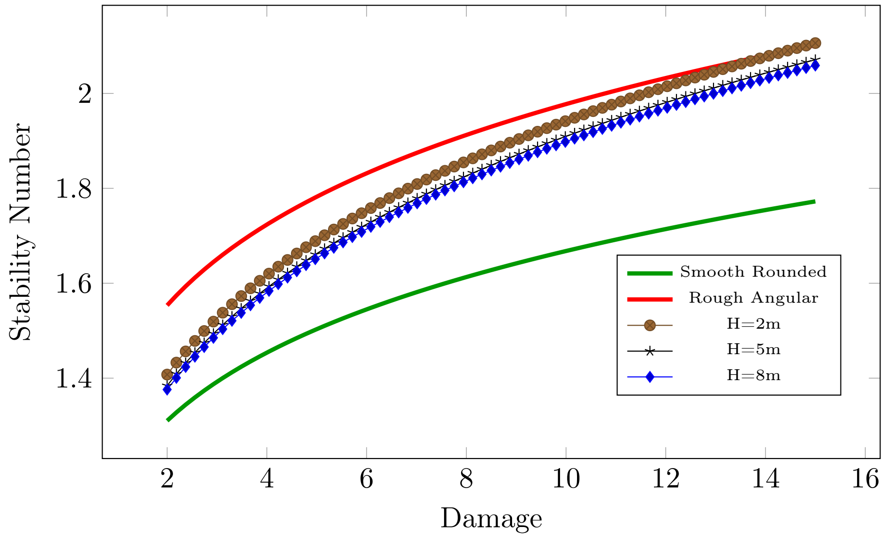


Figure 5.7: N_s *vs* Damage level (S_d), for different wave heights

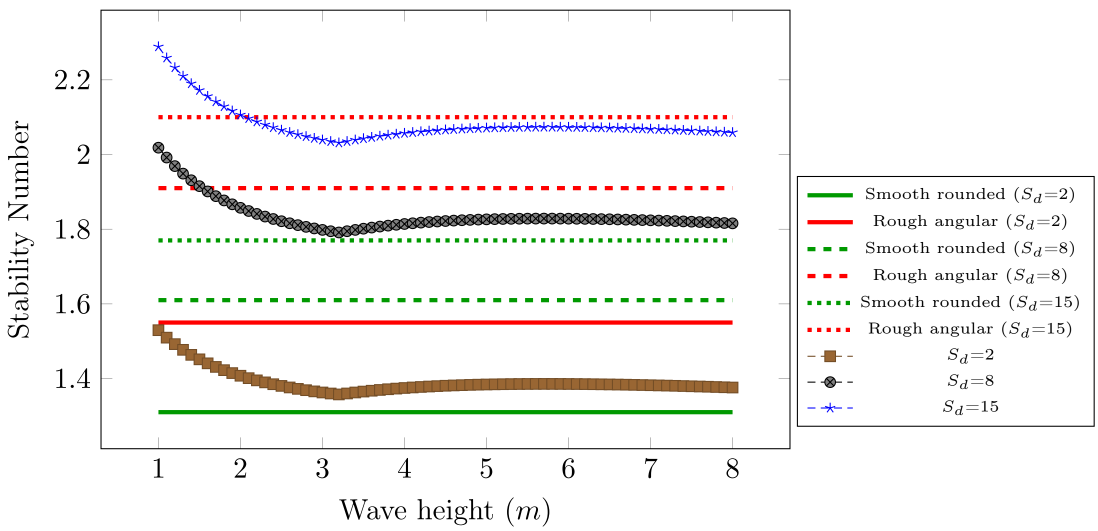


Figure 5.8: N_s *vs* Wave height, for different damage levels

The analysis of data plotted in figure 5.8 shows that, similarly to the storm duration parameter, the absolute difference between two considered damage levels (*e.g.* between $S_d = 2$ and $S_d = 8$) is always the same, regardless of the wave height. This trend is valid throughout all the range of damage level values. The relative difference in stability between the three considered damage level values are calculated using equation 5.10 and presented in figure 5.9.

$$\Delta_{N_s} = \frac{N_{s_{S_{di}}} - N_{s_{S_{dj}}}}{N_{s_{S_{dj}}}} \tag{5.10}$$

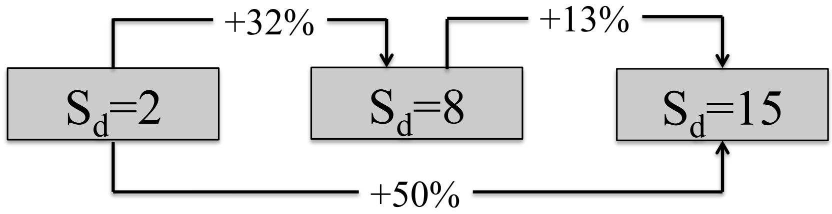


Figure 5.9: N_S variation for different damage levels

Figure 5.9 shows that one unit increase in the damage level has more influence in the armour layer stability for lower damage levels ($S_d < 8$) than for upper damage levels ($S_d > 8$). This is also supported by figure 5.7, as the inclination of the curves becomes more gentle with the increase of the damage level.

In figure 5.8 it is possible to see that for S_d=2 (the characteristic value for the Hudson equation), the Van der Meer equations gives similar results to the modified Hudson equation. For higher damage levels Van der Meer tends to underestimate the stability in relation to the modified Hudson equation. Due to the little use of the modified Hudson equation in design process, further studies should be conducted to validate this approach.

5.2.4 Slope angle

In figure 5.10, the plot lines in the plunging region are steeper than the ones in the surging region, which indicates a larger influence of the slope angle in stability for plunging waves than for surging waves.

Figure 5.11 shows that the Van der Meer equation overestimates the influence of the slope angle in the stability, as the stability number calculated with Van der Meer equation go outside the boundaries created by the recommended values for Hudson equation. However, the Van der Meer formula greatly overestimates the stability, specially for more gentle slopes. The stability number calculated for the 3:4 (V:H) slope angle by the

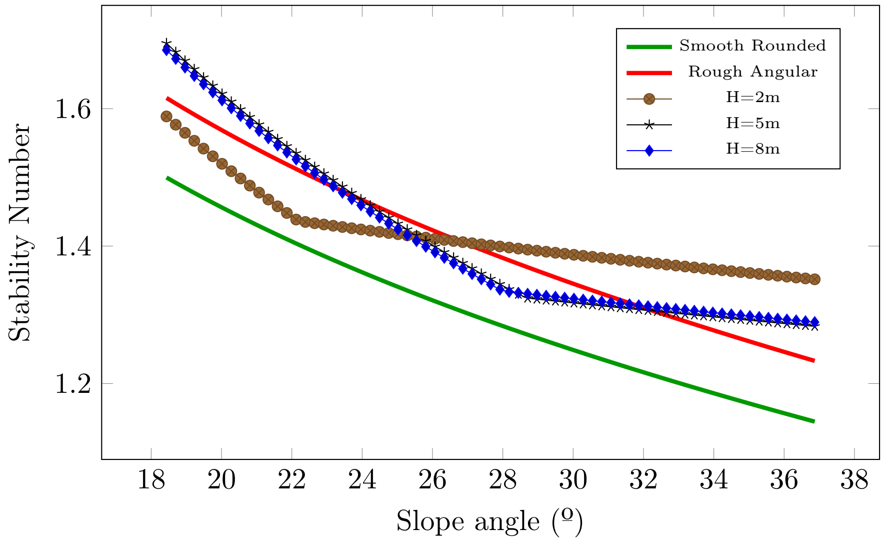


Figure 5.10: N_s *vs* Slope angle, different wave heights

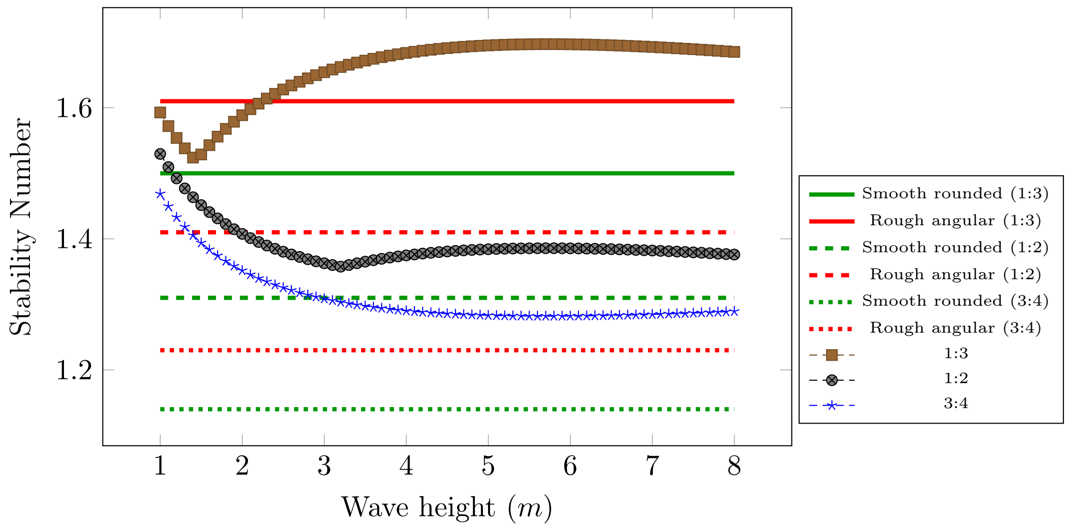


Figure 5.11: N_s *vs* Wave height, for different slope angles

Van der Meer equation is completely outside the recommended values for the Hudson equation.

Using a similar method as used in the assessment of the K_D variation for different values of permeability, the variation for different slope angles was calculated and presented in figure 5.12. In the case of parameters implicit in the modified Hudson formula, this analysis is made using the stability number instead of the stability coefficient (equation 5.11).

$$\Delta_{N_s} = \left(\frac{N_{s_{\alpha i}} - N_{s_{\alpha j}}}{N_{s_{\alpha j}}} \right)_{H_k} \tag{5.11}$$

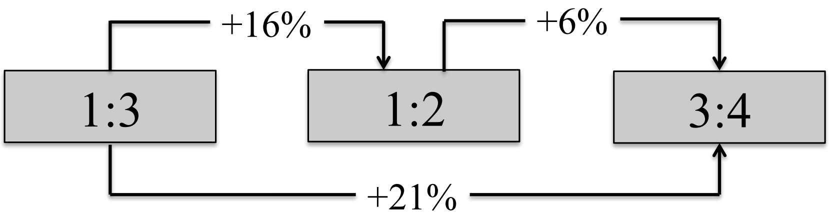


Figure 5.12: N_S variation for different slopes

Figure 5.12 shows that with the decrease in the slope steepness, its influence on the armour layer stability decreases. In this case, the difference is likely to be caused by the transition between plunging and surging waves happening between 1:3 and 1:2.

The weight of the slope angle on the stability does not seem to be influenced by the wave height, as the plot lines for $H_S = 5$ and $H_S = 8$ are nearly overlapping. The influence of the wave height is higher in the transition from plunging to surging waves.

The slope angle is not found to be a very influential parameter by itself, as seen in figure 5.12. However is very conditioning for the transition between surging and plunging waves, which in turn is very influential for other parameters such as permeability.

Chapter 6

Structural design examples

6.1 Angeiras (CONSULMAR, 2011a)

At Angeiras, an intervention was proposed, that consists in two parts:

- Dredging of a 50m wide access canal, to the depth of -1.0m (CD);
- Construction of a 448m rubble mound breakwater, with a stone armour layer and a concrete crest structure.

The objective of the intervention is to provide the area with better shelter conditions for the navigation of fishing boats when on approach, departing and beaching. The implementation area is located in Angeiras, between the Minho and Douro rivers, in the Lavra parish. This location is home to a small fishing community, which will greatly benefit from this intervention. Besides the fishing activity, the tourism in the beach area is the main economic activity of the population. Figure 6.1(a) provides a map of Portugal with the location of the main cities and of Angeiras beach. Figure 6.1(b) provides an overview of Angeiras and the implementation area.

6.1.1 Local conditions

In the design of maritime structures, it is of major importance the knowledge of local conditions, in order to better specify the design of such structures to a specific location.

Shoreline

The shoreline considered for this study can be divided in two different sections, regarding the coastal erosion experienced in past years. The extension of about 5 km north of the intervention site is predominantly rocky seashore, meaning that there has not been a

(a) Portugal (b) Angeiras

Figure 6.1: Map of Portugal with Angeiras beach location

significant change in the shoreline in the last thirty years. In the other hand, the section to the south of the intervention site mainly consists of sandy beaches for about 2 km. In this section, it is estimated that there has been an average erosion of about 60 to 70 meters, in the same time period. In the intervention site, Angeiras beach, there has been an increase of about 15 to 20 meters in the shoreline, between 1965 and 1994, as a result of accretion, after the construction of a breakwater.

The correct study of the sea bottoms and the alluvium regime in the seashore is important to evaluate the impact of the shelter structure construction. The sediment flow reduction results in a greater capacity of sediment transport than availability of sediments, which leads to the erosion phenomenon in order to keep feeding the sediment flow.

Geomorphology

The beach area in which the structure is to be implanted is a small rock promontory that rises about 1 meter from the sand level. From here, the structure extends to about -3.0m (CD) depths, in extremely irregular rocky bottoms. Due to the very energetic wave climate of the coast, the bottoms are always changing, and the cross section of the beach profile alike.

Tides

Accordingly to the tide tables published by the Portuguese Hydrographic Institute, the tide levels in the studied area can be estimated using the typical tide values in Viana do Castelo.

Based on one year of observations using a tide gauge, the Hydrographic Institute predicts that the characteristic tide values for a metonic cycle (19 years), and although these values may change each year within the cycle, the simplified values presented in table 6.1 may be used.

Table 6.1: Angeiras tide levels

Tide	Level (m)
Max HW	+4.10
HHW	+3.5
LHW	+2.7
LLW	+0.50
HLW	+1.40
Min LW	+0.10

Where:

- Max HW and Min LW - Maximum and minimum water level predicted to occur under normal atmospheric conditions, regarding all the possible astrologic conditions;

- HW and HLW - Mean values of water level of two successive high or low tides that occur every 15 days, when the tide amplitude is larger (spring tide);

- LHW and LLW - Mean values of water height of two successive high or low tides that occur every 15 days, when the tide amplitude is smaller (neap tide).

The tide previsions by the Hydrographic Institute are made considering normal atmospheric conditions. However, the event of abnormal atmospheric conditions, such as considerably high or low atmospheric pressure, can cause significant changes to the present values. Considering that minimum and maximum tide values only occur during equinoxes, the probability of simultaneous occurrence of these tides and large alterations to the atmospheric pressure is relatively low, therefore, the presented values can be considered valid.

Wave height

Knowing the extreme values of wave heights on the implementation site is very important to define the structural design of the intervention. These values are normally determined through the interpretation of wave height data, applying one or more statistical distributions. In the present case, the low depths of the implementation area should condition the maximum wave height. However the extreme values of wave height were still extrapolated from available data and then verified if the small depths are truly a limit to the wave height considered in the preliminary design.

The data used to characterize the extreme values of wave height were:

- Hindcast of sixteen storms occurred in the Portuguese west coast, in the period between 1955 and 1981, done by DNV, using the results for a spot between Figueira da Foz and Cabo da Roca;
- Extrapolation done by Pires and Pessanha, from the Meteorology Institute, based on the data from the wave buoy of Cabo da Roca and Sines.

The results from this study are presented in table 6.2.

Table 6.2: Extreme values of wave height

Return Period (years)	H_s (m)		
	D.N.V.	Pires and Pessanha	Mean
5	11.2	9.5	10.4
10	12.1	10.4	11.3
50	14.1	12.4	13.3
100	15.0	13.1	14.1

The values presented on the last column of table 6.2 can be considered as representative of the offshore conditions in the intervention area. However, considering the small depths and the small slope of the bottoms, the bigger waves should break before hitting the implantation zone. In order to calculate the wave heights at the work site, the method proposed by Seeling (1979) was used.

Considering wave periods of 12, 14, 17 and 19s, for a range of depths close to the ones in the implementation zone, the values obtained fot the maximum significant wave heights are presented in table 6.3. These calculations were preformed for the more adverse tide conditions, which is the highest high water, adding an elevation of 0.4m.

As demonstrated in table 6.3, the small depths provide a limit to the maximum wave height in the implementation area, as the offshore wave heights are substantially larger than the ones on the implementation area.

Table 6.3: Maximum significant wave heights at different depths

Bottom depths (m)	Hs max (m)
+1.0	2.6 / 3.0 / 3.6 / 3.7
0.0	3.2 / 3.6 / 4.2 / 4.4
-1.0	3.7 / 4.4 / 5.0 / 5.1
-1.5	4.2 / 4.7 / 5.3 / 5.5
-2.0	4.6 / 5.3 / 5.7 / 5.8
-3.0	5.5 / 6.2 / 6.3 / 6.8
-4.0	6.4 / 6.8 / 7.0 / 7.2

6.1.2 Preliminary design

General conditions

The fact that the implementation area for the structure is located after a series of rocky outcrops, gives it a certain protection, as the barrier formed by these elements causes an early break of the higher waves. This means that the breakwater will only be exposed to a small portion of the wave energy.

In the area right before the head section, the solution was designed with an assumption of the depth, due to a lack of bathymetry data.

In order to minimize the visual impact of the structure, the top level should be limited. As a result, the breakwater allows overtopping of some magnitude and, consequently, stability verification of the inside slopes deserves special attention.

The breakwater should be designed to sustain wave and tide conditions that correspond to a return period of 30 years. The concrete crest is only intended to allow access of technical personal in order to perform maintenance procedures and to access the lighthouse.

Structure description

In figure 6.2 it is possible to see the layout of the structure. The breakwater has a total length of 448m and can be divided into three distinct segments:

- Strait segment with 285m of length, starting in the beach at about +6.0m (CD) and oriented to SW. This segment is implanted mainly in rocky bottom with depths that vary from +2.0m (CD) to -2.5m (CD);

- Curved segment with 103m of length, along which the breakwater turns 30º to south, passing through numerous shoals that rise above the Chart Datum and rocky bottoms with depths of about -2.0m (CD);

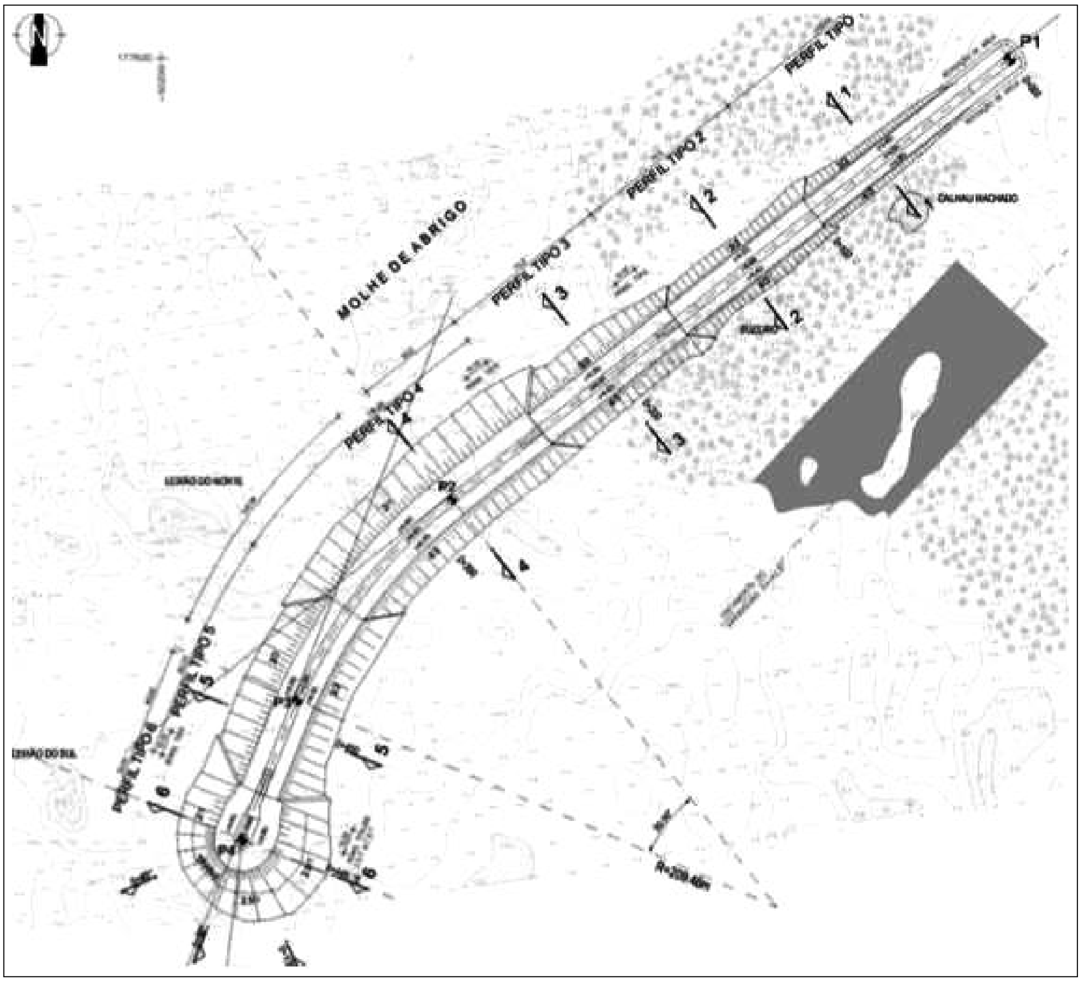


Figure 6.2: Layout of the Angeiras breakwater, adapted from CONSULMAR (2010b)

- Strait segment with about 60m, extends in the direction of SSW and is implanted in rocky bottoms with depths ranging between -1.0m (CD) and -4.0m (CD), ending in the structure's head that is placed on top of a sandbank.

The core of the structure provides a work platform with 6.5m width and +4.0m (CD) of elevation, with slopes ranging from 3:4 to 1:2.5, according to the cross section. The platform is extended to a width of 10m in the head section. The concrete crest structure is 3.5m wide and with an elevation raging from +5.5m (CD) and +6.5m(CD). In the head section, the crest is 10m wide.

Considering the length of the breakwater and the different degrees of exposure to the waves, six different cross sections (figure 6.2) are considered and designed individually.

A point in common to all the cross sections is the use of a two layer of rock armour solution for the protection of both interior and outer slopes and the use of a single layer in both berms. The concrete crest is 3.5m wide throughout the entire length of the structure, except for the head section, in which it widens to 7.0m. A representative cross-section of the structure is presented in figure 6.3.

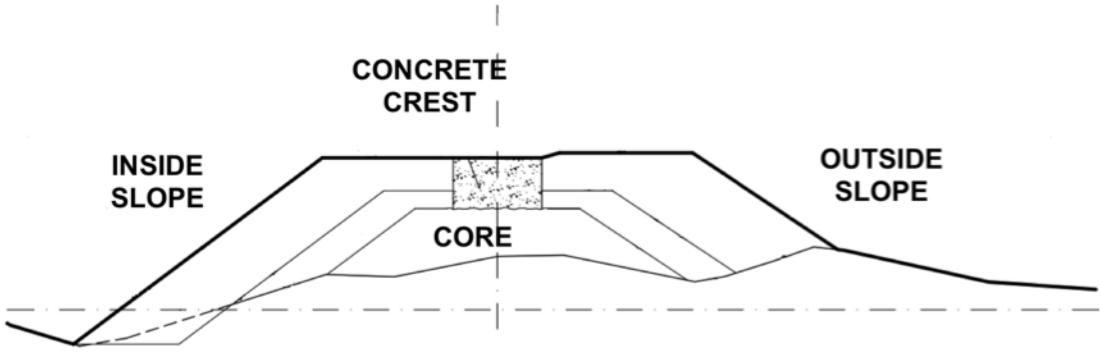


Figure 6.3: Generic cross-section of the Angeiras breakwater, adapted from CONSULMAR (2010b)

Design wave height

The design wave height is the maximum wave height compatible with the depth of each segment of the breakwater. Given the rocky nature of the sea bottom, the depth considered is the one obtained in the latest surveys (January 2000), not considering, for safety reasons, the accumulate of sand that is expected to occur in the north side of the breakwater. The calculation method for the wave height that each section of the breakwater would be exposed was based in the publications Seelig and Ahrens (1980), USACE (2002) and CIRIA *et al.* (2007).

The design wave heights considered for each section of the breakwater are presented in table 6.4.

Table 6.4: Project wave heights

Cross Section	Hs (m)	Direction
1	2.4	Oblique Attack
2	3.0	Oblique Attack
3	3.9	Oblique Attack
4	4.5	Oblique Attack
5	3.9	Frontal Attack
6	5.0	Radial Attack

Stability verification and design

The design of the armour layer of the breakwater was preformed using the Hudson Equation to calculate the weight of the rock elements to place in each section. The main geometric and mechanic parameters used are presented in table 6.5.

Table 6.5: Geometric and mechanical parameters for design

Number fo armour layers	n=2
Slope at the trunk	$\cot\alpha$ = 1:2.5, 1:2
Slope at the head	$\cot\alpha$ = 1:2, 2:3, 3:4
Stability coefficient (K_D)	2.5 (head) to 3.5
Specific weight of rock	$\gamma_r = 26kN/m^3$
Specific weight of salt water	$\gamma_w = 10.25kN/m^3$

The slope angle was corrected considering the angle in which the waves attack the structure. This topic is discussed in section 4.8 of this document.

Considering the difficulty in using only rocks of a particular weight, the weight of the elements to be placed on the protection layer should range between 75 and 125% of the calculated value, providing that more than 75% of the elements have an individual weight bigger than the calculated value.

The calculation of the sub layer weight is based on the weight of the protection layer, using the criteria presented in equation 6.1.

$$\frac{P}{15} = Psm = \frac{P}{10} \tag{6.1}$$

Where:

- Psm - Weight of the element in the sub layer (kN)
- P - Weight of the element in the armour layer (kN)

The thickness of the layer and sub layer are calculated using:

$$r = n \cdot K_{\Delta}^{3} \cdot \sqrt[3]{\frac{P}{\gamma_e}} \tag{6.2}$$

Where:

- r - layer thickness (m)
- n - number of layers
- K_{Δ} - form factor (1.00 to 1.10)

- P - Weight of the element (kN)
- γ_e - Specific weight of the element (kN/m)

The resulting parameters of the individual design of each cross section are presented in table 6.6.

Table 6.6: Stability parameters for each cross section

Cross Section	Length (m)	Crest Elev. (m)	Inner Slope Incl. (H:V)	Inner Slope Elev (m)	Inner Slope Weight (kN)	Outer Slope Incl. (H:V)	Outer Slope Elev. (m)	Outer Slope Weight (kN)
1	100	5.5	4:3	4.9	10 - 20	3:2	5.6	10 - 20
2	70	5.5	4:3	5.6	10 - 30	3:2	6.0	30 - 60
3	70	6	4:3	6.0	30 - 60	3:2	6.2	60 - 90
4	106	6.2	4:3	6.2	60 - 90	2:1	6.5	90 - 120
5	82	6.2	3:2	6.2	60 - 90	3:2	6.2	60 - 90
Head	-	6.5	2.5:1	6.6	120 -150	2:1	6.6	120 - 150

6.1.3 Physical model

Description

In order to validate the design of the structure in terms of stability, overtopping and wave climate in the sheltered area, a physical model at the scale of 1/48 was constructed. The construction of the model and the preforming of the tests was assigned to LNEC. The elements represented in the physical model were:

- Breakwater;
- Coastline;
- Complete bathymetry, around and in front of the structure, up to the depth of -10m (CD), according to the surveys;
- Small inclination slope between the bathymetry of -10m (CD) and the depth of -14m (CD), which is the depth of the wave generators.

Although the model specifications suggested the representation of the bathymetry up to a depth of -20m (CD), this was not possible due to the large distance from the structure to the location of that depth, and to the fact that the use of a higher geometric scale could lead to undesirable scale effects that could contaminate the results.

Scale

One of the disadvantages of physical models is the possibility of model and scale effects. These effects occur when physical properties of the structure can not be scaled properly.

When designing models in marine hydraulics, the main forces are gravity and pressure, surface tension and viscosity can be considered of secondary importance. Considering this, it is common to use Froude's Law to express the relation between the model and the real structure. In this case, the geometric scale was defined as being 1:48. Considering Froude's Law of Comparisson and assuming that the Stability Number (Ns), defined in equation 6.3, is the same in the model and in the prototype.

The Stability number is defined as:

$$N_s = \frac{H_s}{\Delta D_n} \tag{6.3}$$

Where:

- H_s - significant wave height (m)
- Δ - relative volumetric mass density $(\Delta = \frac{\rho_a - \rho_w}{\rho_w})$
- ρ_a - volumetric mass density of rock (kg/m^3)
- ρ_w - volumetric mass density of water (kg/m^3)
- D_n - Diameter of the element $(D_n = \frac{M}{\rho_w}^{-3})$ (m)
- M - Mass of the element (kg)

Since the geometric scale of the model and the material properties are defined, the masses of the individual blocks to use in the model can be determined by the equation:

$$\lambda = \frac{H_{s,p}}{H_{s,m}} = \frac{\Delta_p}{\Delta_m} \cdot \sqrt[3]{\frac{M_p}{M_m}} \cdot \sqrt[3]{\frac{\rho_p}{\rho_m}} \tag{6.4}$$

Where:

- λ - Geometric scale of the model
- $H_{s,p}$ - Significant wave height in the prototype (m)
- $H_{s,m}$ - Significant wave height in the model (m)
- Δ_p - Relative volumetric mass density in the prototype
- Δ_m - Relative volumetric mass density in the model

- ρ_p - Volumetric mass density of the material in the prototype (kg/m^3)
- ρ_m - Volumetric mass density of the material in the model (kg/m^3)

Accordingly to the specifications, the model was constructed using blocks with a volumetric mass density of 2640 kg/m^3, slightly higher than the one in the prototype (2600 kg/m^3). The density of the concrete blocks is the same for the model and prototype (2400 kg/m^3). The density of the water used in the model is of 1000 kg/m^3, slightly lower than the one of salt water (1025 kg/m^3). Considering these parameters and equation 6.4, the mass of the individual model blocks is presented in 6.7, along with the corresponding weight in the prototype.

When conducting studies in scaled models involving fluid dynamics, the viscosity effects should be considered if the model and the prototype have different cases of fluid flow. In order to ensure that the fluid flow regime in the armour layers of the model is turbulent, as in the prototype, the following condition should be satisfied:

$$R_e = \frac{\sqrt{g \cdot H_s \cdot D_n}}{\nu} > 3 \cdot 10^4 \tag{6.5}$$

Where:

- R_e - reynolds number
- g - gravity acceleration (m/s^2)
- ν - kinematic viscosity (m^2/s)
- H_s - wave height (m)
- D_n - diameter of the element (m)

Table 6.7: Mass and weight of individual blocks

Block	M_p (kN)	M_m (g)
Rock	10 - 30	8 - 23
	30 - 60	23 - 46
	60 - 90	46 - 69
	90 - 120	69 - 92
	120 - 150	92 - 116
Antifer	100	81

Considering an average weight of $90kN$, with a correspondent diameter of $1.52m$, the viscosity effects can be considered insignificant for $H_s > 4.5m$. Considering that the conditions for the stability tests indicate wave heights higher than $4.5m$, the Reynolds Number criteria is satisfied for most of the structure. In some of the smaller blocks, some scale effects are expected, which are not possible to eliminate due to the restrictions to the model scale caused by the limited size of the tank. However such small effects do not jeopardize the test results.

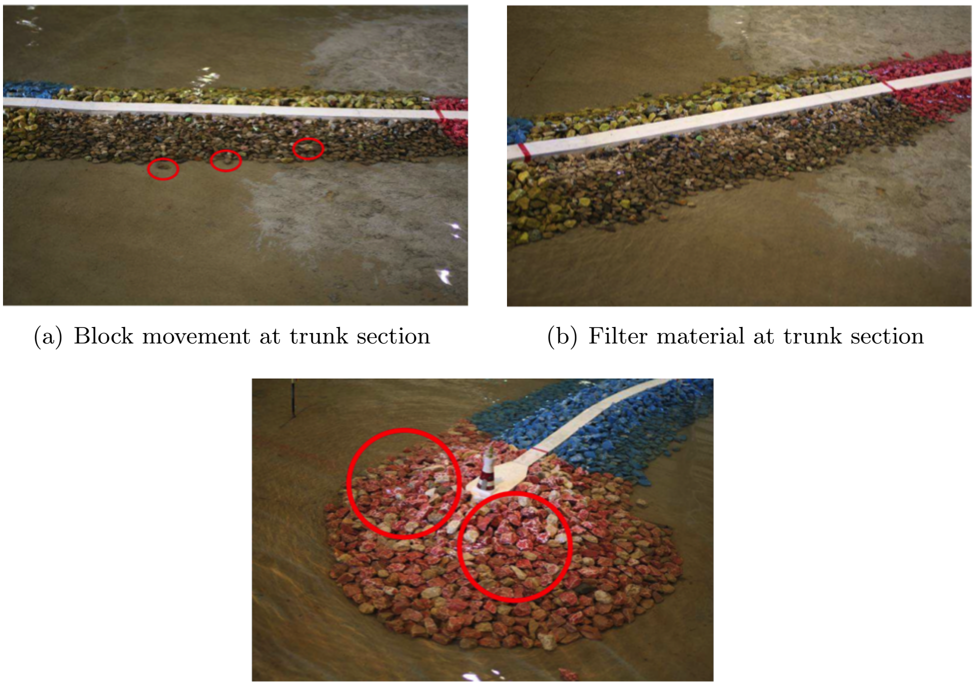
Test program

The stability tests on the physical model were planed in order to test the structure's response under a different combination of factors. The tide levels were the Maximum High Water (+4.1m) and the Minimum Low Water (+0.1m). The initial wave periods to be considered were 12s and 18s, however, during the calibration phase, it was found that using a wave period of 18s, the equipment could not produce the desired wave heights. Considering this limitation, LNEC and CONSULMAR agreed to lower the higher wave period to 16s, in order to maintain the desired wave heights, with values ranging from 3m to 7.5m. The duration of the test for each combination of tide level, wave period and wave height was 3 hours.

Test results for the Base Solution

When testing the base solution, three situations were observed:

- Some blocks from the outer slope of cross section 1 and 2 were moved to inside area of the structure (figure 6.4(a));
- Some filter and core material started to appear in the inside slope of cross section 2 and 3 (figure 6.4(b));
- Some blocks were removed in a very precise area of the head section, creating two small craters in the armour layer (figure 6.4(c)).



(a) Block movement at trunk section

(b) Filter material at trunk section

(c) Damage to head section

Figure 6.4: Test results for Base Solution

Alternative Solution 1

In response to the results obtained in the first series of tests on the physical model, LNEC and CONSULMAR agreed to introduce some alterations to the design, in order to improve stability. The changes were as follows:

- Replacement, in the head section, of the rock armour (figure 6.5(a)), by Antifer blocks with 100kN of weight (figure 6.5(b)). The blocks are placed in two layers, keeping the same slope inclination and filter materials as the original design;
- Increase in the elevation of the concrete crest to +6.5m (CD) in the entire structure. In the original design, only the head section had this elevation;
- Replacement of the 10 to 30kN rock armour (figure 6.5(c)), in the inner slope of cross section 2, with 30 to 60kN rock armour (figure 6.5(d)).

(a) Base Solution Trunk

(b) Alternative Solution Trunk

(c) Base Solution Head

(d) Alternative Solution Head

Figure 6.5: Alterations to the structural characteristics of Base Solution

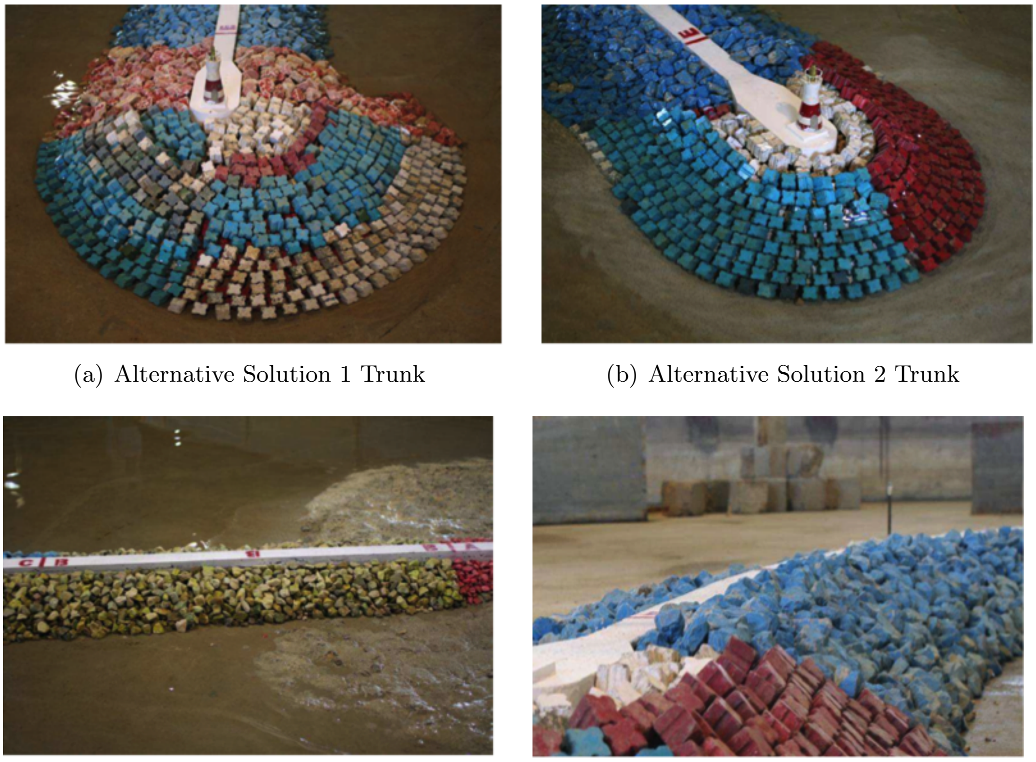
Test results for Alternative Solution 1

When testing the alternative solution 1, there was confirmation that the problem with the removal of blocks from the berm and the lost of filter and core material were resolved. In the head section, some movement of the blocks of the head section were detected, however this was not enough to jeopardize the overall safety of the structure.

Alternative Solution 2

In order to further improve the stability and lower the cost of the structure, CONSULMAR recommended a second solution that includes the following changes:

- Change in the slope inclination of both inner and outer slope of the head section to 3:2, creating a symmetric cross section (figure 6.6(a) to figure 6.6(b)). This change, if proven stable, allows the structure to be more cost-effective, by needing less material to construct the head section;
- Placement of a second layer of rock armour, on the berm of the inner slope, throughout the entire length of the structure (figure 6.6(c) to figure 6.6(d)).



(a) Alternative Solution 1 Trunk (b) Alternative Solution 2 Trunk

(c) Alternative Solution 1 Head (d) Alternative Solution 2 Head

Figure 6.6: Alterations to structural characteristics of Alternative Solution 1

Test results for Alternative Solution 2

When testing the alternative solution 2, some block movement was still detected on the berm of the head section, but the results were satisfactory since this design allows to save a large quantity of material in the construction of the head section.

6.2 Velas Harbor (CONSULMAR, 2013)

The Velas harbor is located in the south coast of the S. Jorge island, Azores (figure 6.7). The main objective of the harbor is to provide a platform for the commercial navigation that supplies the island. In addition, fishing boats, for refueling purposes, frequently use the harbor.

The commercial harbor was recently remodeled and received major improvements, with emphasis on the construction of a harbor for recreation proposes, the creation of a $1360m^2$ pavement area for cargo storage and a new access road from the harbor to the island's road system.

The objective of this project is to extent the length of an existent jetty, which includes

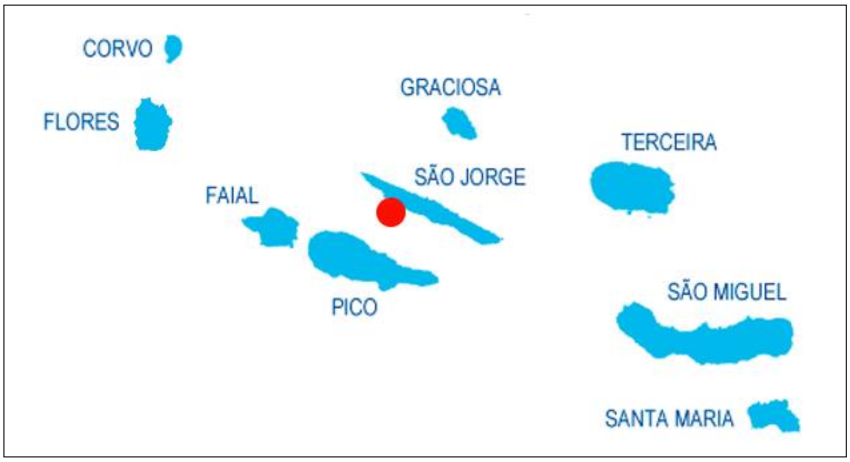


Figure 6.7: Map of Azores (the implementation area is marked with a red dot)

a landing platform, enabling the possibility of receiving larger ships and a larger storage area for containers, increasing, at the same time the shelter provided for the fishing and recreation infrastructures located in the inside.

The proposed intervention consists in five parts:

- Demolition of the concrete structure and removal of the armour blocks from the existing structure to provide foundation and reuse in the new structure;
- Dredging and excavation to provide a suitable foundation;
- Extension of the existing protection slope;
- Extension of the existing landing platform;
- Construction of a new lighthouse and extension of the existing technical areas.

6.2.1 Local conditions

Geomorphology

The bathymetry of the bottoms in the intervention area was obtained by the overlapping of the results of surveys conducted in 1995, 1999 and 2010. Accordingly to these elements, the bathymetry of the bottoms follows the profile of the island's shoreline, reaching the depth of -10m (CD) at 100m of the coast and -16m (CD) in the head section of the existing jetty. After this section, the bathymetry of the bottoms continues to follow the profile of the shoreline, which, at this point, is parallel to the direction of the intervention. The new structure is expected to have foundation at depths ranging from -18m (CD) to -20m (CD).

The existing harbor structures are founded on top of a rocky bank that extends in a north-south direction. The geologic survey of the implantation area was preformed using underwater seismic methods and a side-scan sonar. These surveys show that the bottoms are mainly rock covered by a thick layer of sand, as indicated by the regularity in the bathymetry.

Tides

In the Azores archipelago, the tidal constituent is the principal lunar semi-diurnal, with mean amplitude of 0.9m and maximum amplitude of 1.8m. Accordingly to the predictions of the Portuguese Hydrographic Institute, published in the tide tables between 1982 and 2010, the characteristic tide values in Velas harbor are presented in table 6.8.

Table 6.8: Velas Harbor tide levels

Tide	Level (m)
Max HW	+1.94
HHW	+1.65
LHW	+1.27
LLW	+0.73
HLW	+0.37
Min LW	+0.17

Where:

- Max HW and Min LW - Maximum and minimum water level predicted to occur under normal atmospheric conditions, regarding all the possible astrologic conditions;
- HW and HLW - Mean values of water level of two successive high or low tides that occur every 15 days, when the tide amplitude is larger (spring tide);
- LHW and LLW - Mean values of water height of two successive high or low tides that occur every 15 days, when the tide amplitude is smaller (neap tide).

The present tide values were obtained considering the registry of tide levels in the Horta harbor and applying the necessary corrections to the amplitude for the Velas harbor. There should be an addition of +0.1m to these values, to account for the rise of the sea level in the following decades. The tide previsions by the Hydrographic Institute are made considering normal atmospheric conditions. However, the event of abnormal atmospheric conditions, such as considerably high or low atmospheric pressure, can cause significant changes to the present values.

Wave height

The location of the Velas harbor, in the south coast of S. Jorge island, results in a excellent natural shelter for the agitation coming from Northwest. The Pico and Faial islands provide additional shelter from the agitation coming from the Southwest and South-southeast. However, the harbor is still exposed to the agitation generated by local winds.

In order to account for the influence that the other islands produce in the wave propagation, two separate wave types were considered:

- Type A - Waves generated by local winds, normally associated to smaller periods;
- Type B - Waves generated at greater distances, normally associated to large periods.

In order to characterize the offshore agitation, the hindcast made by the United Kingdom Meteorological Office of a point south of the Azores central group for a period of 25 years. These elements were used in numerous studies and confronted to with real data, proving to be reliable.

In order to design the extension of the structure, it is necessary to know the wave heights at the implementation area. Using software developed by CONSULMAR, the offshore wave height values were corrected for two points representing the harbor entrance (P1), at the depth of -30m (CD), and the edge or Velas bay (P2), at the depth of -50m (CD).

6.2.2 Preliminary design

Design wave height

For the stability verification of maritime structures, the wave height is associated to a return period of 50 to 100 years and, on the overtopping verification, the have height is associated to a smaller return period of 5 to 10 years. In order to obtain the maximum wave height values it is necessary to extrapolate the existing data to the mentioned return periods.

Based on the wave height values obtained to each mentioned point, the maximum wave heights were grouped considering the two main directions (East and West), and the type of wave. These values were then extrapolated to return periods from 1 to 100 years, using the Gumbel distribution, given by the formula:

$$F(H) = \exp[-\exp(-\alpha(H-\beta))] \tag{6.6}$$

Where α and β are parameters that can be determined directly from the data sample.

This data treatment produced the results presented in table 6.9 for each point, direction and wave type:

Table 6.9: Maximun values of wave height

Return period (years)	P1 (-30m CD)				P2 (-50m CD)			
	West direction		East direction		West direction		East direction	
	Type 1	Type 2	Type 1	Type 2	Type 1	Type 2	Type 1	Type 2
1	3.1	2.2	1	0.5	4.3	3.2	1.8	0.9
2	3.5	2.4	1.2	0.6	5.1	3.6	2	1.1
5	4.1	2.7	1.3	0.8	6.1	4.2	2.3	1.4
10	4.5	2.9	1.4	0.9	6.8	4.6	2.6	1.6
20	5	3.2	1.5	1	7.6	5	2.8	1.7
50	5.6	3.5	1.7	1.2	8.6	5.5	3.2	2
100	6	3.7	1.8	1.3	9.3	5.9	3.4	2.2

Considering the importance of the wave height in the design process, a second extrapolation was conducted on the direction that demonstrated to be the most adverse (West), using the Weibull distribution, given by the formula:

$$P(H) = 1 - \exp\left[-\left(\frac{H-a}{b}\right)^{c}\right] \tag{6.7}$$

Where "b" and "c" are parameters that can be determined graphically and "a" is location parameter, obtained by performing iterations.

Table 6.10 presents the results from this data treatment, along side with the respective values resulting from the Gumbel distribution and the mean value between them.

In an attempt to minimize possible errors due to approximations, the wave height value for the return period of 50 years was corrected considering a confidence interval of 90%. The resulting value was a wave height of 6.5m.

Table 6.10: Comparisson between values obtained by diferent destributions for P1

Return period	Destribution		
(years)	Gumbel	Weibull	Mean
1	3.1	3.5	3.3
2	3.5	3.9	3.7
5	4.1	4.4	4.2
10	4.5	4.7	4.6
20	5.0	5.1	5.0
50	5.6	5.6	5.6
100	6.0	5.9	6.0

Structure description

The breakwater should be extended by a length of 150.9m and keep the level of the landing dock. The outer slope has an inclination of 3:2 (H:V) throughout all the structure length, except for the head, which has an inclination of 2:1. The head section rotates 180 degrees. The structure core, except for the head section, has a core with an inner slope of 4:3 inclination and an outer slope of 3:2. The core forms a berm at the depth of -15m (CD) with a slope with 4:3 inclination, protected by a sub-layer of rock armour with weight ranging from 10 to 30kN and a protection layer made of 300kN Antifer blocks. The slope ends at the top berm, at the elevation of +7.2m (CD). The transition of the new and existing profile is made within the first 20m of the extension. In the last 20 meters of the structure body, before the head section, the profile is gradually changed from 3:2 to 2:1. A representative cross-section of the structure is presented in figure 6.8.

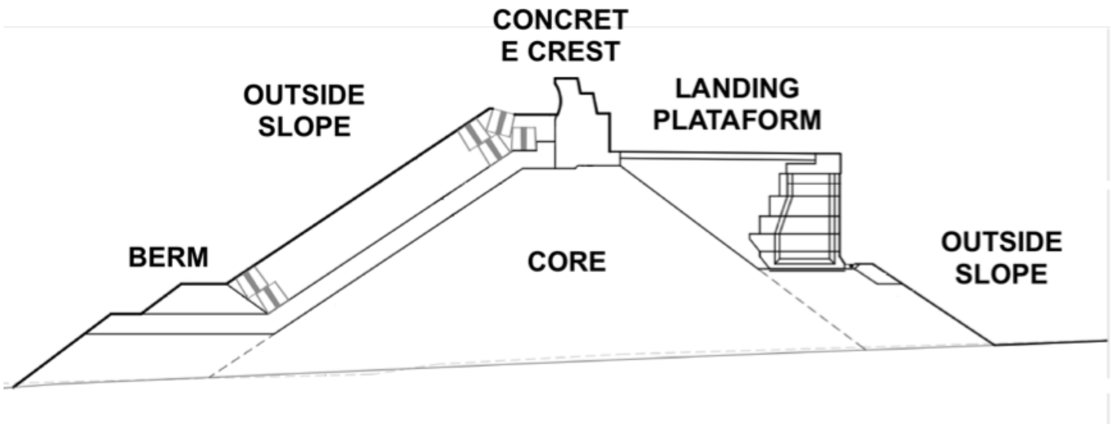


Figure 6.8: Generic cross-section of the structure, adapted from CONSULMAR (2010a)

Stability verification and design

The design of the protection layer was preformed using the Hudson and Van der Meer equations (eq. 6.9), that provide diferent aproaches to the Stability Number (eq. 6.8), to calculate the weight of the blocks to place. The type of block considered was the one that already protects the structure and was to be removed for the construction works (Antifer).

$$N_s = \frac{H_s}{\Delta D_n} \tag{6.8}$$

$$N_s = \begin{cases} (K_D \cot\alpha)^{\frac{1}{3}} \\ \left(6.7\frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \end{cases} \tag{6.9}$$

$$\Delta = \frac{\gamma_r}{\gamma_w} - 1 \tag{6.10}$$

$$D_n = \sqrt[3]{\frac{W_r}{\gamma_r}} \tag{6.11}$$

Where:

- H_S - Wave height (m)
- γ_r - Specific weight of the element (kN/m)
- γ_w - Specific weight of salt water (kN/m)
- W_r - Weight of the element (kN)
- K_D - Stability coefficient
- α - Slope angle
- N_{od} - Relative damage level
- N - Number of waves
- s_{om} - Slope of the fictitious wave ($S_{om} = \frac{H_s}{L_{om}}$)
- L_{om} - Length of the offshore wave (m)

Considering a project wave of 6.5m and Nod=0.5, the weight of the Antifer is 300kN.

6.2.3 Physical model

Description

In order to validate the design of the structure in terms of stability, overtopping and wave climate in the sheltered area, a physical model at the scale of $1/51$ was constructed. The construction of the model and the performing of the tests was assigned to LNEC. The elements represented in the physical model were:

- Complete bathymetry, around and in front of the structure, up to the depth of -30m (CD);
- The original structure and the structures present in the inside of the harbor, especially the recreation harbor;
- Extension of the structure (figure 6.9).

Test program

The test program planned for the physical model considers two different water levels, two different wave attack directions and a series of combinations between wave height and period. An overview of the test program is presented in table 6.11.

Table 6.11: Test program on the Velas harbor physical model

Test series (-)	Test (-)	Water leve (m)	Wave direction (-)	Period T_p (s)	Wave height H_S (m)
1	1_3	0	W	8	3, 4, 5
	4_8			10	3, 4, 5, 6, 7
	9_15			12	3, 4, 5, 6, 7, 7.5, 8
	16_32			14	3, 4, 5, 6, 7, 7.5, 8, 9
2	1_3	+2	W	8	3, 4, 5
	4_8			10	3, 4, 5, 6, 7
	9_15			12	3, 4, 5, 6, 7, 7.5, 8
	16_32			14	3, 4, 5, 6, 7, 7.5, 8, 9
3	1_2	0	SSE	8	2, 3
	3_4			10	2, 3
	5_8			12	2, 3, 3.5, 4
4	1_2	+2	SSE	8	2, 3
	3_4			10	2, 3
	5_8			12	2, 3, 3.5, 4

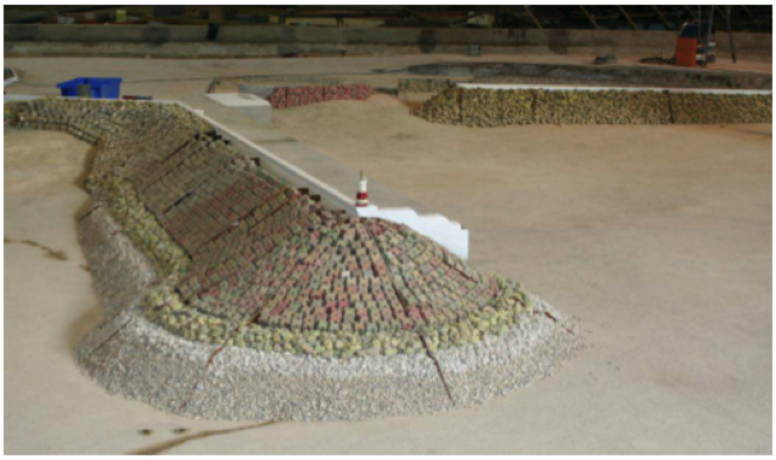


Figure 6.9: Physical model of the extension of the Velas harbor structures

Test results

The protection layer of the design structure has proven perfectly stable in all test conditions, exception being the bottom line of Antifer blocks. These blocks revealed a small degree of instability, as a result of a defective berm. The berm on which the protective layer lies suffered some damages in the tests with higher wave heights and period values. The two affected zones that were the inside of the head section, where the head connects to the landing dock, and where the profile is gradually changed from 3:2 to 2:1.

6.3 Discussion of the practical examples

For the purpose of discussing the applicability and validity of the Hudson stability coefficient, the laboratory and design data provided by CONSULMAR (2011a) and CONSULMAR (2011b) was analyzed.

In the case of the Angeiras intervention, the analysis started by applying a coefficient to the structure's slope angle in order to take into account the wave incidence angle in relation to the structure armour layer. The information about the main direction of the wave propagation at the site was taken from table 6.4. According to the table, waves attack the structure perpendicularly at section 5. From this premiss, looking at CONSULMAR (2010b), the wave incidence angle was found for the remaining sections of the structure. Exception was made for the head section, in which a normal attack was considered, due to this being a round section that should be attacked by a normal direction wave at some point. The bases for applying the reduction coefficient to the slope angle were described in section 4.8.2. The wave attack angles and the modified slope angle are presented in table 6.12.

Table 6.12: Slope angle modification values in Angeiras

Cross Section (-)	Wave attack angle (degrees)	Slope angle (H:V)	Mod. slope angle (H:V)
1	27.00	3:2	3.37:2
2	27.00	3:2	3.37:2
3	27.00	3:2	3.37:2
4	23.00	2:1	2.17:1
5	Normal	3:2	-
Head - Outer	Normal	2:1	-
Head - Inner	Normal	2.5:1	-

Applying the Hudson equation (equation 3.1) to the variables presented in CONSULMAR (2011a) in table 6.5 and 6.4, the weight of the rock units to place in the external slope of each of the cross sections was calculated and compared (see table 6.13) to the results obtained presented in CONSULMAR (2011a) that were used later in the physical model tests. Table 6.13 shows that only the calculated weight for sections 4 and "Head - outer" diverge from the values presented by CONSULMAR (2011a). In the case of section 4, the calculated value underestimates the weight of the unit, in relation to the range of weights considered in the design. This is probably because of an overestimation of the wave incidence angle coefficient to this section of the structure. This can be due to the uncertainty in the measuring of the angle in the process described earlier, if the wave attacks the structure in a more perpendicular way, the weight of the blocks would be in the design range. As for the case of the outer head section, the weight is overestimated and physical model tests were performed with smaller weight blocks, to test their hydraulic performance. This is a common practice in the design of breakwaters, due to the uncertainty given by the empirical formulas, especially in the head section of the structure. If the solution with the smaller weight proves to be reliable in the physical tests, the final solution will be more cost-effective than considering the larger value at first.

A similar analysis can be performed using the weights presented by CONSULMAR (2011a) to calculate the correspondent K_D values and compare to the reference values found on the bibliography. This comparison is presented in table 6.14 and, in line with the weight comparison, the only section whose values do not fit in the range of the recommended values is the outer head section.

Table 6.13: Comparison between calculated and reference weights in Angeiras

Cross Section (-)	H_s (m)	K_D (-)	Mod. Slope angle (H:V)	Weight (kN)	Reference weight range (kN)
1	2.4	3.5	3.37:2	15.2	10 - 30
2	3.0	3.5	3.37:2	29.8	30 - 60
3	3.9	3.5	3.37:2	65.4	60 - 90
4	4.5	3.5	2.17:1	77.8	90 - 120
5	3.9	3.5	3:2	73.4	60 - 90
Head - Outer	5.0	2.5	2:1	162.4	120 - 150
Head - Inner	5.0	2.5	2.5:1	129.9	120 - 150

In response to the results of the tests performed in the physical model, changes were made to the armour layer characteristics. These changes are described in section 6.1.3. The more interesting changes for this discussion are the change in the armour unit, in "Alternative solution 1", and the change in the inclination of the structure's slope in "Alternative solution 2". These changes require a new analysis of the considered stability coefficient.

The comparison on stability coefficients of Antifer units should be performed with close attention due to the high range of reference values found in the bibliography (see section 4.1). The reference values presented for Antifer units in table 6.15 are taken from Freitas *et al.* (2013) for uniform placement for two different packing densities. Due to the lack of information on placement type and packing densities in the studied structure, a better analysis was not possible.

Table 6.14: Range of K_D values correspondent to the weight results from the design process

Cross Section (-)	H_s (m)	Weight range (kN)	Mod. Slope angle (H:V)	K_D range (-)	Ref. K_D values[a] (-)
1	2.4	10 - 30	3.37:2	5.3 - 2.7	4.0 - 2.1
2	3.0	30 - 60	3.37:2	3.5 - 1.7	4.0 - 2.1
3	3.9	60 - 90	3.37:2	3.8 - 2.5	4.0 - 2.1
4	4.5	90 - 120	2.17:1	3.0 - 2.3	4.0 - 2.1
5	3.9	60 - 90	3:2	4.3 - 2.9	4.0 - 2.1
Head - Ext.	5.0	120 - 150	2:1	3.4 - 2.7	2.8 - 1.7
Head - Int.	5.0	120 - 150	2.5:1	2.7 - 2.2	[b]

[a] Considering USACE (1977), values for a two layer armour constructed with smooth or round rocks and attacked by breaking or non-breaking waves (see tables 4.1 and 4.2);
[b] There are no reference values for this slope inclination on the head section.

Table 6.15: Range of K_D values correspondent to the changes made in the structure of the Angeiras breakwater

Solution (-)	Section (-)	Slope (H:V)	Unit (-)	Weight range (kN)	Kd range (-)	Reference values (-)
Base	Outer	2:1	Rock	120 - 150	2.7 - 2.2	2.8 - 1.7[a]
	Inner	2.5:1	Rock	120 - 150	3.4 - 2.7	-[b]
Alternative 1	Outer	2:1	Antifer	100	3.2	5.8 - 4.0
	Inner	2.5:1	Antifer	100	4.1	5.8 - 4.0
Alternative 2	-[c]	3:2	Antifer	100	5.4	5.8 - 4.0

[a] Considering USACE (1977), values for a two layer armour constructed with smooth or round rocks and attacked by breaking or non-breaking waves (see table 4.2);
[b] There are no reference values values for this slope inclination on the head section;
[c] In "Alternative solution 2" the inner and outer slope have the same inclination.

In the case of breakwater in Velas harbor, there were no reduction to account for the wave angle incidence, therefore, normal wave attack was considered. The structural parameters provided by CONSULMAR and considered for this analysis, were:

- $H_s = 6.5$m
- $W_{unit} = 300KN$
- Slope inclination = 3:2 (H:V)
- $\gamma_r = 25kN/m$ (assumed value)
- $\gamma_w = 10.05(kN/m)$

Using this variables and considering Hudson's equation, the resulting K_D value is 4.6 which, accordingly to Freitas *et al.* (2013) is in the normal range for a double layer of Antifer units with regular placement.

Chapter 7

Final remarks

7.1 General comments

The main focus of this dissertation was to study and increase the understanding about the factors and coefficients that are characteristic to the empirical formulations on which coastal engineering is dependent. The subjectivity related to the coefficients estimation is an obstacle because, although some guidance can be found on reference manuals, some parameters and its influence on stability are difficult to characterize. Therefore, design of coastal engineering projects still requires knowledge based on the experience of the designer or design team.

The work developed in this dissertation on coastal engineering, particularly on the assessment of structural stability in coastal structures, has developed the knowledge over design methodologies and intervening parameters.

This chapter presents a summary of the developed work, with emphasis on the main conclusions and recommendations for future developments.

7.2 Conclusions

The response to a problem involving coastal engineering is often not straightforward. Normally there are numerous solutions for a problem and, for every solution, the balance between benefits and disadvantages should be considered. This document presents the most frequently used structures in coastal engineering, together with the respective pros and cons indispensable in the decision making of each solution to consider.

In order to compare different design solutions (different types of structures, profiles, layer definitions), it is necessary an accurate definition of the stability. The definition considered in this document is the *stability number* and the *stability coefficient*.

In the project of coastal structures, the design of the armour layer is of great importance to ensure that the structure fulfills its purpose for the desired return period. The fact that the structure is design for a certain return period, does not mean that it will not sustain any damage during its life time. A probabilistic approach to the design should be considered, otherwise it would result in a over-costly structure. This probabilistic design approach acts on the definition of the design wave. This parameter was already discussed in Marinho (2013) and, therefore, is not addressed in this document.

The design process of coastal structures normally passes through the application of empirical formulations to the considered local conditions and the test of the resulting solution by means of a scaled model in a wave tank. This process could be considered iterative, the designed solution may be proven unreliable and new design and physical model tests would be necessary. The necessity to test solutions using physical models comes of the fact that the existing empirical formulations do not offer enough confidence, due to the actual impossibility of one formulation to take into account the large number of governing variables that influence stability. The focus of this document is on the two most widely used empirical formulas for the armour layer stability assessment. These are the Hudson formula and the Van der Meer formulas.

The Hudson formula is the most widely used due to its simplicity and proven results from decades of use. This formula makes use of the *stability coefficient* parameter to take into account all the parameters that affect the structure and are not implicit in the formula. This approach has the advantage of not being strict and allowing a great number of variables to be considered through recommended values for the stability coefficient. However, this advantage, rises the problem of the lack of recommended values that take into account some important parameters like porosity, damage level, storm duration and type of wave attacking the structure.

The Van der Meer formula is a more recent and complex formula that has the advantage of accounting for the above mentioned variables that are not considered in the Hudson formula. On the other hand, it is a closed formula that does not allow for any other variable to be considered. Moreover, the original formula is only valid for structures with a two layer armourstone protection and located at relative high depths. In order to use a different armour unit or a relative low depth location, a variations of the original formula must be considered. Also, this formulation does not cover important parameters such as interlocking capabilities of different types of stones and the unit location within the structure (head or trunk section).

From the assessment of both formulas, it becomes clear that the best candidate to account for a larger number of variables is the Hudson equation through the stability coefficient. Due to the lack of test data that would allow a better understanding on the influence of each of the variables and derive the correspondent stability coefficient value, the Van der Meer formula was used to determine the stability coefficient variation with each of the considered parameters.

In order to allow comparison between both formulas, a modified Hudson equation was used, that takes into account the damage level. A consequence of using the Van der Meer equations to determine the parameter influence on the stability coefficient is that all the parameters (with the exception of the storm duration) are sensible to the type of wave attacking the structure (plunging or surging).

In the case of permeability, the influence on K_D is considerably large and not linear. Instead, it was found highly dependent on the wave height or period. It was concluded that for the considered conditions (P=0.4), the recommended stability coefficient values by USACE (1977) for the stability coefficient are conservative, in the case of smooth round stones, and underestimate the stability, in the case of rough angular stones.

The storm duration parameter is found independent or with very little dependency on both the wave height and the type of wave attacking the structure. The influence of this parameter on the stability coefficient value decreases as the number of wave increases.

Both the damage level and the slope parameters are included in the the modified Hudson formula and, therefore are not included on the stability coefficient. However, it does not mean it is completely independent from them. Each of the considered formulations treats the parameters differently and for that reason, the comparison between them is interesting. In the case of the damage level, it appears to have more importance in the Van der Meer approach. Generally, the modified Hudson equation is more conservative and results in larger armour units. The influence of this parameter is found to be much larger between the *no-damage* and the *intermediate* damage conditions than between *intermediate* and *failure* damage conditions. For the *no-damage* condition, the stability is found to be according to the recommended values. As the damage level rises, the Van der Meer approach tend to result in similar values as the Hudson with the recommended value for rough angular stones, which could result in a overestimation of the layer stability and smaller units than necessary.

In both formulations, the slope angle of the structure appears to have a smaller influence on stability than the other studied parameters. However, in the Van der Meer formulas, besides having a direct influence on the stability of the layer, the slope angle also influences greatly the type of wave attacking the structure. Therefore, influences all other parameters that depend on the type of wave attacking the structure.

Although the trends in the parameters influence are valid, the stability comparisons presented in this document, between the two formulations, are only valid for the described Van der Meer conditions and the interdependency between the parameters is not considered.

The wave attack angle is an important parameter that is not covered in any of the considered formulations. Various studies regarding this parameter were considered and some trends are noticeable:

- The influence of wave attack angle on the stability of armour is significant and required unit size can be reduced compared with perpendicular wave attack, which is the worst situation;
- The uncertainty related to the wave attack angle definition has a larger influence on layer stability for intermediate attack angles and is smaller for small and large attack angles;
- The influence is dependent on the type of armour unit;
- The dependency on other parameters such as permeability and wave steepness are not consensual, as some studies point to a dependency and other reject the same dependency.

Various formulations to account the wave attack angle are presented in this document, nevertheless this influence should be always tested in physical model tests.

In the study cases provided by CONSULMAR, specially in the case of the Angeiras project, it is possible to see the iterative between designing and testing the structure in order to find the most cost-efficient solution, even for an experienced team. A more precise formulation could prevent the necessity of the extra round of tests and design.

7.3 Future developments

Considering that the results from existing formulations for the assessment of layer stability are associated with a relatively large uncertainty, there is room for improvements in the accuracy. The Hudson formula in particular, for being the most used, it would be worthy of studies to improve the confidence in the results.

As seen in this document, the Hudson formula, through the stability coefficient value, is able to consider additional parameters that influence the armour layer stability and are not considered in the recommended K_D values. In order to analyze the influence of each parameter, physical model tests are recommended. Varying the test conditions, it would be possible to understand how each parameter influences the layer stability. An analyzes of the influence of parameters on other parameters (*e.g.* the influence of the wave period on permeability) is also important to accurately describe the parameter.

Alternatively, the reanalyzes of the test data from previous studies, could be a less expensive option. Sometimes the results from studies with a different intent can also be used, through a different interpretation of the data. Moreover, the review of data from studies with the same intent could result in different conclusions and new findings.

The final target for these analyzes would be to, if possible, reduce the subjectivity factor in stability coefficient and replace it with the influence of the included parameters. New revisions on the K_D coefficient would allow a better description of the stability conditions.

One of the main advantages of the Hudson formula is its simplicity of use and, in order to maintain this quality, new recommended values should be created. If some of the parameters that are corrently missing, were to be integrated, we would be faced with a more accurate formulation.

Not only the exposed layer, but also other elements of the coastal structure should have their stability evaluated. One important element is the toe of the structure, which represents an important role in the overall structural behavior of the structure.

Ultimately, coastal engineering is still a field where subjectivity plays a significant role, due to the complexity of the different factors and the interaction between them. For this reason, there is still progress to be made.

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